Califor
Regional
'acility

2.98 3.50 June 25, 1906

"World" Building. American Tract Society. "Times" Building. Park Row Bldg. N. Y. Post-Office. St. Paul Building. Home Insurance Bldg.

View of lower New York from Dun Building.

Frontispiece.

View of Post-Office Square, New York.
The Park Row Building, in the Centre of this Illustration, is the highest Office Building in the World.

Frontispiece.

ARCHITECTURAL ENGINEERING.

WITH ESPECIAL REFERENCE TO

HIGH BUILDING CONSTRUCTION,

INCLUDING MANY EXAMPLES OF

PROMINENT OFFICE BUILDINGS.

BY

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SECOND EDITION, REWRITTEN.

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PREFACE TO REVISED EDITION.

THE author has endeavored, in the following pages, to define and illustrate, in a manner as practicable as possible, such of the fundamental principles in the constructive design of modern high buildings as may prove useful to architects, engineers, and students.

While the technical press of the country and the transactions of various architectural and engineering societies have contained a great number of admirable papers and addresses on many of the individual subjects here considered, yet the realization of the want of some practical and comprehensive collective data on the subject of steel building construction has induced the writer to rewrite and extend this volume.

Before the present revision was undertaken, the author's descriptions and examples were mainly limited to Chicago practice, as previous to the year 1894 Chicago stood prominently first in the construction of high buildings. Since then, however, New York and other Eastern cities have begun to rival and even outstrip Chicago in this form of building, while the same interval has witnessed great changes and improvements in many details of construction, especially in terra-cotta and concrete floors, and indeed in all methods of fireproofing. Many notable tests of fireproofing methods have also occurred since the publication of the first edition of this work, such as the tests of various floor constructions by the New York Building Department, the test of methods afforded by the burning

of the Pittsburg buildings, and the still more recent partial destruction of the Home Life Building in New York.

Fireproofing is so intimately related to steel building construction that the author has been tempted to include in this revision considerable matter given in more logical sequence and in more detail in his volume on "The Fireproofing of Steel Buildings," published by John Wiley & Sons, New York, 1899; but in order to avoid repetition, and to keep somewhat within former limits, the subject of fireproofing has been introduced only as it is necessary to a proper understanding of the design and calculation of the framework.

The largely local character of previous illustrations has been supplemented by notable examples in different localities so as to make the scope more general than formerly. Where previous examples have been found still to remain illustrative of the best practice, they have been retained as representing principles rather than the very latest examples. An effort has also been made to exclude such data as are given so admirably by the handbooks of the various steel companies, but the student or architect is earnestly advised to supplement such points as may be found of interest in this volume by the more detailed tables and illustrations in the many valuable handbooks now issued.

The following chapters are arranged in the order in which the calculations for such structural work must proceed, starting with the load-bearing floor system, thence through the successive stages to the foundations. The latter would seem to require the first attention; but as they are the last to be calculated, being dependent on all other considerations, they have here been placed last. The illustrations and examples given have been largely obtained through the courtesy of the architects of the respective buildings. An endeavor has been made to present only the most practical methods.

J. K. F.

BOSTON, October, 1901.

CONTENTS.

CHAPTER I.

CHAPTER II.

CHAPTER III.

CHAPTER IV.

CHAPTER IX.

CHAPTER X.

LIST OF ILLUSTRATIONS.

ARCHITECTURAL ENGINEERING.

CHAPTER I.

"SKELETON" OR "CAGE" CONSTRUCTION.

SKELETON construction is a natural outgrowth resulting from conditions imposed upon the owners of property lying within the business sections of our large American cities.

The fact that in large communities it is found most advantageous as to time and convenience for business transactions, to have all possible office buildings and commercial interests concentrated within limited areas, has caused the adoption of buildings of such heights as were not considered possible, and still less practicable, before the introduction of steel-skeleton methods.

Topographical limitations have also proved potent factors in this tendency toward concentration. In New York City the business and financial centre has long been established within a comparatively limited area at the extreme end of Manhattan Island, and extension in area meant a growth in one direction only, thus undesirably increasing the length of the channels of business intercourse; while in Chicago, where limitations of area were first overcome, the commercial centre covers only three-fourths of a square mile within topographical boundaries which make enlargement impossible.

The erection of high buildings with greatly increased floor space thus became a necessity, not only as an accommodation for the rapid growth of trade interests, but as a business proposition in the improvement of real estate so situated. Increased floor areas became necessary to insure a realization on the investment, and with the enormous and seemingly ever-increasing values of real estate in the centres of such limited commercial areas, the natural vertical extension of floor upon floor has constantly increased in the endeavor to make investment in such buildings a safe and profitable business venture.

The high building has become a fixed and definite feature in our large American cities at least, and its inception and growth have been made possible only through the introduction and rapid development of steel-building construction and fire-proofing methods. Without steel buildings, the art of fire-proofing would never have been called into existence, while without the development of fire-proofing principles, steel construction, as applied to buildings, must have been discontinued long ago.

Early Forms of Iron Construction.—All forms of iron and steel construction have undergone wonderful changes in comparatively recent years, and there are few fields where more radical growth and improvement may be noted. To appreciate this, it is only necessary to remember that most of our present forms and combinations of rolled iron and steel were unknown in either bridge or building practice fifty years ago, and a comparison of present types with early examples of cast- and wrought-iron in building work reveals many decidedly interesting curiosities. Take, for example, the iron girders removed from Sedgwick Hall at Lenox, Mass., some years ago. (See Fig. 1.) "These were each made of three plates, a top and a bottom one, both horizontal, with a vertical corrugated web plate between, the corrugations running up and

down. The three pieces were fastened together with vertical bolts extending through the top and bottom plates, about 20 ins. apart, and alternating, one on this side and the next on the other side of the vertical plate, the transmission of strains

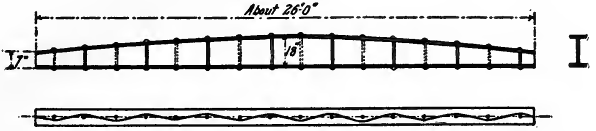


FIG. 1.—Early Form of Wrought-iron Girders, used in Sedgwick Hall, Lenox, Mass.

from the web to the flange depending entirely upon friction. These beams were probably placed in position about 1840, and some of them still remain in the building." *

The strange forms employed in cast-iron were well illustrated in some cast-iron floor-beams removed from the old Boston Public Library in 1899. Fig. 2 was made from

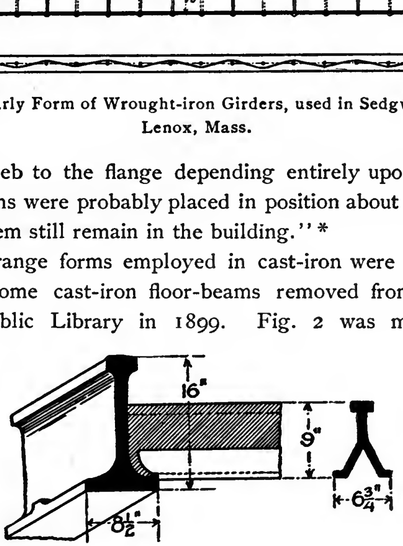


FIG. 2.—Cast-iron Girders and Floor-beams removed from Old Boston Public Library.

sketches and photographs taken by Mr. C. H. Blackall, the architect of the new Colonial Building erected on this site. The cast-iron girders, 16 ins. deep, were spaced about 10 ft. centres. The floor-beams or joists, spaced about 4 ft. centres

* See "The Use of Steel in Large Buildings," by C. T. Purdy, *Journal of the Assoc. of Eng. Societies*, vol. xiv. No. 3.

and carrying segmental brick arches, were hooked over lugs cast on the lower flanges of the girders, as shown in the illustration.

Introduction of Present Forms and Methods.—With the invention of the iron I-beam in France and England in 1853, the manufacture of floor-beams was at once introduced into this country. Iron I-beams were first rolled in the United States at Trenton, N. J., in 1854, while steel beams were not rolled until as late as 1885, when their manufacture was started by the Carnegie Steel Co.

Iron, as a substitute for wood in constructive purposes, was long thought to be fire-proof, or fire-resisting, because incombustible. For this reason, iron not only replaced wood in many features of building construction, but was also used as a substitute for masonry, as is shown by the extended use of cast-iron for entire fronts of buildings from about 1855 to 1870. This practice was principally due to the idea that cast-iron, because incombustible, was superior to marble or stone work, which would crack and flake when exposed to fire and thus be ruined in appearance, even though it did not fail completely. A few cases of total failure, however, in such cast-iron fronts, and the discovery that iron became unreliable under temperatures of 1000° Fahr., caused such construction to be condemned by fire departments and insurance interests, and after about 1870 the necessity of some adequate protection for constructive iron members became generally recognized. And no sooner did fire protection as a covering for structural steel become an established fact, than the development of iron and steel forms and combinations progressed hand in hand with those improvements in fire-proofing methods which furthered development and encouraged originality in this field.

Previous to 1883, a height of nine or ten stories was very nearly a practical limit in building construction. Beams and columns usually formed an adjunct only to the masonry, as

the walls were made heavy enough to carry the floor-beams and girders, and the resultant loads. Circular cast-iron or Phœnix columns of wrought-iron were used for interior supports, while the floor arches, if intended to be fire-proof, were usually made in segmental form of brick or corrugated iron, or terra-cotta at later dates, levelled on top with concrete to the finished floor lines. On the introduction of iron for constructive purposes, in 1854, the only fire-resisting material known was ordinary brick work, and fire proof floor construction was obtained through the use of segmental brick arches, sprung between the beams. Corrugated iron and concrete floors were then introduced, in an effort to dispense with the centering required for brick arches. Both of these methods, however, were heavy, clumsy, and expensive as compared with present types, but they formed the only standards until the introduction of terra-cotta.

In buildings of this character, iron, where employed at all, was used with little or no view toward securing a closely related or interdependent assemblage of component parts. Columns and beams were incorporated in the design in a disjointed, hap-hazard fashion, leaving the principal reliance for stability or strength upon the masonry construction. This naturally limited the possibilities of building design to the safe unit stresses applicable to masonry, resulting in the large piers made necessary for any considerable height, and in bulky foundations of dimension stone which soon reached a limit of spreading area, besides filling up much valuable basement room.

Origin of Steel Footings.—The first radical step toward improvement from the older methods was in the use of iron members to stiffen offsets in concrete foundations. In the Montauk Block, ten stories, built in Chicago in 1881–2 by Burnham and Root, Architects, the foundation piers were made of layers of concrete 18 ins. thick, on top of which were

placed dimension stones forming pyramids, thus nearly filling the entire basement. Under two stacks of fire-proof vaults, such foundations would have interfered with basement space where it was desired to locate boilers and engines, so that, under these conditions, the innovation was adopted of embedding iron rails in the concrete footings to increase the allowable offsets in the concrete layers. This constituted a most important precedent, which has gradually developed into present grillage design.

Origin of Skeleton Methods.—By far the most important step, however, in the development of the Chicago construction, occurred in 1883, when Mr. W. L. B. Jenney prepared plans for a ten-story office building for the Home Insurance Company; and to this architect belongs the credit for the conception of skeleton construction. His departure from previous practice was bold and progressive, and from the successful carrying out of his plans may be dated the marked interest taken in a construction which needed but little stimulus to insure its general adoption.

In order to obtain a maximum light for the offices proposed in his new design, Mr. Jenney decided to reduce the width of all exterior piers as much as possible, and to use cast-iron columns within the piers to carry the floor-loads, thus relieving the masonry piers of these loads, and consequently reducing their areas. The question then arose as to the supposed expansion and contraction of continuous metal columns 150 ft. high, subjected to a variation of some 120° Fahr., and this suggested carrying the walls, as well as the floors, story by story on the columns, thus dividing the movement. The exterior piers were made self-supporting, but the spandrel portions, between the top of one window and the bottom of the window above, were carried on iron girders placed in the exterior walls and extending from column to column. The foundation piers in this building were made of masonry,

pyramidal in form and consisting of alternate courses of rubble and dimension stone.

This method of supporting the walls as well as the floors and floor-loads on beams and columns was a most important departure from former methods, and attendant responsibilities of design were at once encountered. The concentration of superstructure weights resulting from such design was soon given consideration, and the employment of iron rails in foundations, as in the Montauk Block, was extended to more important use in the calculation of isolated footings.

As early as 1872 a pamphlet had been published by Frederick Bauman, entitled "The Method of Constructing Foundations on Isolated Piers," but, excepting the partial adoption of these principles in the Montauk Block, it was not until the same architects designed the Rookery Building in 1885–6, that isolated footings were really employed with the use of steel members. In this building the footings were made of two courses of steel rails laid at right angles to each other and embedded in concrete, with I-beams crossing the upper courses, on which were placed cast column bases. The masonry walls were self-supporting.

Development of Skeleton Methods.—These improvements in design were quickly appreciated, and soon incorporated in succeeding buildings, the Tacoma Building, 14 stories high (Chicago, Holabird & Roche, Architects), being probably the first complete type of skeleton construction. A spandrel section in this building is illustrated in Fig. 3, while one of the column footings is shown in Fig. 4. It is interesting to compare this spandrel section used for carrying a plain brick wall, with the more elaborate spandrel sections shown in Chapter VI, where moulded and ornamental terra-cotta is employed.

From this point on, buildings rapidly improved on their predecessors in matters of detail, and the Chicago or skeleton

type soon became well established. It was found that if the concentrated column loads were properly distributed over the available ground area, the weight of the structures, and conse-

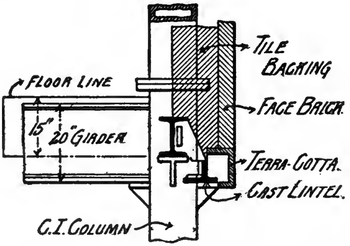


FIG. 3.—Spandrel Section, Tacoma Building, Chicago.

quently the heights, could go on increasing until the footings covered the entire site, within permissible limits of bearing capacity per square foot. Lighter building materials were consequently employed, and buildings were made higher, until,

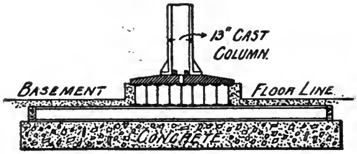


FIG. 4.—Foundation Detail, Tacoma Building, Chicago.

in 1890, the first twenty-storied building (the Masonic Temple) was erected in Chicago.

This revolution of old methods went on simultaneously in both the West and the East, but building laws, industrial interests, and a greater conservatism in the East, retarded there the early development which became particularly marked in Chicago.

The Manhattan Life Building in New York was the first notable example in the East of a building erected after the

new methods, and this structure demonstrated to New York the great possibilities afforded by skeleton construction. The number of floors has gradually increased year by year, reaching a height of thirty stories in the Ivins Syndicate or Park Row Building, while proposed structures of forty stories are now discussed with less astonishment than were twenty floors ten years ago. Nearly all of the ultra-high buildings are now confined to New York City, where the character of the rock foundations, coupled with unrestricted building regulations as to height, make such extreme examples possible.

Skeleton Construction Defined.—"Skeleton construction" very properly defines the type of building construction to which it was at first applied. This suggests a skeleton or simple framework of beams and columns, dependent largely for its efficiency upon the exterior and interior walls and partitions which serve to brace the structure, and which render the skeleton efficient, much as the muscles and covering of the human skeleton (to borrow a comparison used by various writers) make possible the effective service of the component bones. Skeleton construction is thus defined by the present Chicago Building Ordinance:

"The term 'Skeleton Construction' shall apply to all buildings wherein all external and internal loads and strains are transmitted from the top of the building to the foundations by a skeleton or framework of metal. In such metal framework the beams and girders shall be riveted to each other at their respective junction points. If pillars made of rolled iron or steel are used, their different parts shall be riveted to each other, and the beams and girders resting upon them shall have riveted or bolted connections to unite them with the pillars. If cast-iron pillars are used, each successive pillar shall be bolted to the one below it by at least four bolts not less than three-fourths inch in diameter, and the beams and girders shall be bolted to the pillars. At each line of floor- or roof-beams,

lateral connection between the ends of the beams and girders shall be made by passing wrought-iron or steel straps across or through the cast-iron column, in such manner as to rigidly connect the beams and girders with each other in the direction of their length. These straps shall be made of wrought-iron or steel, and shall be riveted or bolted to the flanges or to the webs of the beams and girders."

"If buildings are made fire-proof entirely, and have skeleton construction so designed that their enclosing walls do not carry the weight of floors or roof, then their walls shall be not less than twelve inches in thickness; and provided, also, that such walls shall be thoroughly anchored to the iron skeleton; and provided, also, that wherever the weight of such walls rests upon beams or pillars, such beams or pillars must be made strong enough in each story to carry the weight of wall resting upon them without reliance upon the walls below them. All partitions must be of incombustible material."

"Cage" Construction.—The more advanced and approved practice, however, partakes more of the character of a single unit so far as the steel is concerned, for the framework is now made complete in itself, like a wire cage, and independent of any considerations as to aid or support from any external coverings; hence the name "cage" construction has been applied to this method of high building. The steel framework, originally introduced to carry vertical loads only, has been gradually developed and systematized as increased attention has been bestowed upon the questions of lateral strength and stiffness against wind or other external forces. The use of a well-braced frame now permits the substitution of curtain or veneer walls for the solid masonry construction formerly required, and the reduction in thickness of such walls to 12-in. or 16-in. protective veneer walls only, makes it possible to obtain much larger window areas, besides giving large gains in rentable floor areas. It is also possible to omit heavy

interior walls, substituting therefor light movable partitions which may be placed as desired by tenants. Foundations are now required for large concentrated column loads, instead of distributed loads as formerly, and the problem is more definite, if not more simple, and the design is always considered with reference to securing all available basement or sub-basement areas.

Cage construction, therefore, as exemplified by the best examples, consists of a steel framework with well-riveted beam and girder connections, efficiently spliced column joints, and efficient wind-bracing, to secure its independent safety under all conditions of loading and exposure.

Architectural Engineering.—Architectural engineering, or the application of engineering principles to architectural design and construction, would properly constitute a treatise of great range, including the underlying principles of all building construction, and the strengths of all building materials. But, as the modern office building presents almost, if not quite, all of the *ordinary* problems involved in architectural engineering, this type of construction alone will be considered in the following pages, as being representative of the ordinary requirements demanded of the architect in constructional practice. Special forms of construction, such as complicated foundation problems, involving elaborate cantilever design, or pneumatic caissons, etc., as well as roof trusses or the special trussing over large unobstructed areas, must be generally intrusted to the professional engineer, and more specific data and theory may be obtained from special works on such subjects.

The following chapters are aimed to present, as clearly and practicably as possible, such data and practice as will be found of value in considering such questions as floors and floor-framing, columns, foundations, and other interdependent factors in constructive building design.

CHAPTER II.

FIRE PROTECTION.*

BEFORE considering the details of skeleton construction it will be well to consider the general subject of fire-proofing, with its effectiveness and its limitation.

The total fire loss in the United States during the year 1894 was about $128,000,000, of which the insurance companies paid, as their share, some $81,000,000. This stupendous drain on the resources of the nation may be better appreciated if we consider that the full value of the pig-iron production for the same year was about $75,000,000.

When to this fire loss we add the estimated amount necessary to maintain the fire departments, and to sustain the fire insurance companies, the grand total will exceed $175,000,000 annually.

If, then, it is true, as stated by underwriters that forty per cent. of all fires are attributable to causes easily prevented, a proper treatment of the fire problem certainly becomes a very practical and economic inquiry.

The subject of proper fire protection is now recognized as a legitimate and important branch of engineering. It is no longer confined exclusively to endeavors to protect human life, but is greatly increasing in scope, demanding very careful thought from its economic standpoint as well. And that this

* For more detailed information pertaining to tests and methods of fire-proofing, see author's "Fire-proofing of Steel Buildings," 1899. John Wiley & Sons, N. Y.

question of fire waste is being seriously considered in all its aspects and by all classes of society is shown by the widening facilities for the use of fire-proof construction. The realization of low prices in the building market has served to overthrow many of the hitherto unquestioned prejudices in regard to fire-proof construction, and the economy of such design as opposed to the fire-trap methods so long in vogue is now being daily emphasized by architects, engineers, and the technical press. And what is most gratifying is the fact that this economy is beginning to be appreciated not only by the owners of large office buildings and stores, but also by the more limited investor, as is evidenced by the start already made in fire-proofing the ordinary city house, at a figure but slightly exceeding the cost of non-fire-proof methods. It was found, in taking figures for a building in Philadelphia to cost $125,000, that a thoroughly fire-proofed construction would cost only 3.6 per cent. more than the ordinary method of building. This increase would be compensated for in a very short time by the decreased insurance.

It does not seem unreasonable to hope that fire-proof buildings may soon be the rule, rather than the exception, and that the near future may see all of our mercantile, manufacturing, and even dwelling houses, except those of the very cheapest, built according to fire-resisting principles. Steel, the clay products, and cement or concrete are permanent, fire-resisting, of ready adaptability, and of remarkably low cost. The fire-trap timber construction with its susceptibility to dampness, drought, heat, and cold, involving dry-rot, as shown by the collapse some years ago of a prominent hotel in Washington, must give way to new conditions and further improvement in a field of such promise. The insurance burden will be gradually lightened, and human life be better protected.

Fire-proof Construction.—While buildings *could* be erected with absolutely no inflammable material in their construction,

there would still remain the contents or property of the tenants to feed possible fire. This element of danger cannot be eliminated; and added to this are the dangers that come from without as well as from within. For as long as highly inflammable buildings surround even the most excellent of modern fire-proof structures the term is misleading. Fire-proof structures must stand in fire-proof cities. Hence the word "fire-proof," as applied to a modern structure, does not mean one that claims immunity from all danger of fire, for considerable woodwork must still be used in interiors, and the average contents are dangerous in the extreme; but it does claim to embody principles which have reduced the fire hazard, both interior and exterior, to a minimum, according to the best skill and judgment of the day. The term implies that all structural parts of the edifice must be formed entirely of non-combustible material, or material which will successfully withstand the injurious action of extreme heat.

Following is the definition given in the new building ordinance of Chicago: "The term 'fire-proof construction' shall apply to all buildings in which all parts that carry weights or resist strains, and also all stairs and all elevator enclosures and their contents, are made entirely of incombustible material, and in which all metallic structural members are protected against the effects of fire by coverings of a material which must be entirely incombustible and a slow heat-conductor. The materials which shall be considered as fulfilling the conditions of fire-proof coverings are: First, brick; second, hollow tiles of burnt clay applied to the metal in a bed of mortar, and constructed in such manner that there shall be two air-spaces of at least three-fourths of an inch each by the width of the metal surface to be covered, within the said clay covering; third, porous terra-cotta, which shall be at least two inches thick, and shall also be applied direct to the metal in a bed of mortar."

Chicago Athletic Club Building Fire.—The success that has attended past efforts in fire-proofiing may be judged by such examples of fire as have been afforded in protected structures. One of the earliest and most interesting tests of the new methods was the burning of the Chicago Athletic Club building while under construction. Though not entirely satisfactory as a test of present building methods, "this building furnishes an assurance that was lacking before—that the metal parts of a building if thoroughly protected by fire-proofing, properly put on, will safely withstand any ordinary conflagration, if the quantity of combustible materials the building contains is not greatly in excess of that which enters into the construction of the building itself."

This extract from the report of experts employed to investigate this fire and its effects emphasizes two very important facts, namely, the danger of the indiscriminate use of combustible material not absolutely necessary in the construction, and second, the evident superiority of terra-cotta as a fire-proofing substance.

The above fire, which occurred on November 1, 1892, was the first case on record of a fire in a building intended to be fully fire-proof where the loss to the insurance companies was more than thirty per cent. of its value. It is further stated in the report that "if the building had been completed, it would never have contained combustible material enough (or so distributed) to have produced sufficient heat to have done any considerable damage to the building by burning."

The fire in question was of very intense heat, inasmuch as a vast quantity of scaffolding, flooring, trim, etc., was collected in mass, preparatory to use; but, in spite of this, there seemed no reason for questioning the integrity and strength of the building, as a whole, after the fire, and no doubt existed that the fire-proofing around the columns saved them from utter collapse, because it remained in place until the fuel that had

fed the flames was well-nigh exhausted. The result to the building included the entire destruction of all the interior finish, plastering, piping, and wiring, as well as parts of the elaborate front of Bedford stone and pressed brick. But the steel columns and beams were uninjured, except a few of the latter where unprotected; and the tile arches, built after the end-construction method, were almost uninjured, in spite of the combined action of great heat and frequent applications of cold water.

Pittsburg Fire.—A second notable fire of great importance to all those interested in fire-proofing methods occurred in Pittsburg in May, 1897. This fire resulted in the complete destruction of a large wholesale grocery house, where the fire originated, and in the partial wrecking of three adjoining buildings, all of which were presumably of modern fire-resisting design. Of the latter structures, one was known as the Horne store building, one as the Horne office building, and the third as the Methodist building.

The Horne store and office buildings, of six and four stories respectively, were separated from the Jenkins building by a street 60 ft. wide, but upon the falling of the walls of the latter structure, their unprotected fronts were subjected to the full force of the flames, and the almost complete destruction of the buildings and their contents soon followed. The Methodist building, separated from the Jenkins building by an alley, was not threatened until the side wall of the Jenkins building fell, as, previous to that time, the iron shutters on the Jenkins building side wall had stayed the flames. The Methodist building was, therefore, not subjected to such a severe test as the others, the damage being confined to the destruction of its contents without serious injury to constructive features.

The Horne store building, built in 1893, was a steel-frame structure, with front and rear self-supporting walls. The front windows were of large area and unprotected, while a light-well

extended from the first story to the roof, thus forming a most convenient means of communication of fire from floor to floor. The floor construction consisted of 9-in. hard-burned terra-cotta arches, side-construction, and the columns were protected by 2-in. hard-burned terra-cotta, $\frac{1}{2}$ in. thick, with one air-space. The roof framing consisted of 10-in. beams, without arches, with a suspended ceiling beneath, and covered by light tees at right angles to the beams, to receive 2-in. hollow book-tile. A compression tank, about 6 ft. in diameter and 25 ft. long, weighing, when filled, about 52,000 lbs., was supported by steel beams resting upon the roof girders, and as this entire construction was protected from the upward rush of fire by the suspended ceiling alone, the inevitable falling of the tank soon occurred, with great damage to the steelwork and fireproofing. The loss to the steel frame was about twenty per cent. of its original value, but the appraiser's report stated that the damage to the steelwork would not have exceeded five per cent. of its entire cost, had not so much destruction been wrought by the falling of the water tank.

The brick fronts were seriously injured by the cracking of the stone, and the fireproofing throughout had to be replaced, save a salvage of $16\frac{2}{3}$ per cent. The tops of the hard tile arches were generally found to be in good condition, but the under surfaces were largely broken away, leaving hollow spaces visible from the rooms below. The skew-backs and girder-casings were also badly broken.

The Horne office building was also a steel-frame building, with self-supporting walls. The floor construction was made of 9-in. end-construction porous terra-cotta arches, while the columns were protected by 1-in. solid porous terra-cotta covering blocks. The partitions were also of porous tile.

The contents and wood trim were pretty thoroughly consumed, but the steel construction twas apparently little injured, so that it was not uncovered for examination. The

entire loss to the fireproofing, excepting the partitions, was $33\frac{1}{3}$ per cent. of the entire cost, while the partitions suffered a loss of some 43 per cent., this being largely due to the use of wood nailing-strips which burned out and allowed the partitions to fall. The bottoms of the porous terra-cotta arches were but little broken, and the column coverings generally remained intact.

In the Methodist building, the floor arches or slabs, built after the Metropolitan system, made a satisfactory showing, but the test was not severe enough to furnish any positive deductions.

The Pittsburg fire, in brief, served to furnish additional proof of a most conclusive character that steel buildings which are properly protected by *porous* terra-cotta, with brick or terra-cotta exterior walls and properly constructed interior terra-cotta partitions, may, if reasonable consideration is given to provisions against the internal spread of fire or the external communication of fire, be confidently relied upon to fulfil all reasonable requirements.

Fig. 5 shows an exterior view of the two Horne buildings, after the fire, while Fig. 6 is an interior view of the Horne Store building.

Home Insurance Building Fire.—The value of fireproof construction was further demonstrated by an exposure fire of considerable note which occurred in the Home Insurance Building in New York City on Feb. 11, 1898. This 15-story skeleton structure was well designed against internal hazard, but the burning of a highly combustible adjoining building subjected it to an exposure which it was unable to withstand, and the upper floors were considerably damaged by the entrance of the up-rushing flames into the side and court windows. The Home building is of modern steel-frame construction, with a self-supporting front wall of white marble. The floor arches were of hard tile, side-construction, while the

FIG. 5.—Exterior View of Horne Store and Office Buildings, Pittsburg, after Fire.

Fig. 6.—Interior View of Horne Store Building, Pittsburg, after Fire.

column casings and partitions were of porous tile. The damage, except by water, was confined to the upper floors, consisting principally of the destruction of the marble front above the eighth floor, and the falling of the terra-cotta partitions, due to their being built upon the wood flooring in many cases, and with wood door- and window-frames. The floor arches stood the test remarkably well, the action of the column covering was very satisfactory, and the structural steel was but slightly damaged.

It is not to be claimed that any of these examples have proved entirely satisfactory as ideals of fireproof construction, but certain underlying facts have been clearly proved by these tests; and taking these essential points as a basis for further improvement, and using the utmost care and judgment in the matter of general design and details, it must be recognized that the use of terra-cotta, as seen in the best examples of recent fireproof buildings, offers a successful solution of one of the most important problems of modern times.

Introduction of Tile or Terra-cotta.—Hollow tile as a building material was first introduced in the United States in 1871, shortly after the great Chicago fire. Its first use was for floor arches, to replace the old brick-arch method. Terra-cotta was a direct outcome from conditions imposed by the increased height, and hence weight, of a rapidly developing architectural construction, and its necessity was doubtless made more apparent by the great object-lesson afforded by Chicago's disastrous conflagration. A substance was necessary to replace the heavy masses of masonry which constituted the fireproofing at that date, both in the exterior walls and in floor arches, and the peculiar advantages of terra-cotta caused it to undergo many improvements in rapid succession, affecting not only its use in floor construction and column and beam protection, but adapting it to the needs of a lighter and more rapid construction throughout. Its attendant reduction in

weight, its great fire-resisting qualities, its peculiar adaptability to all conditions of position and form, its susceptibility to modelling, and its readiness of manufacture in shapes convenient for transportation and erection, soon caused it to win favor both for its artistic possibilities and its enduring qualities, through which it becomes one of our most valuable constructive media. First used in interior work only, it soon appeared in belt courses, sills, caps, ornamental panels and modelled work in the hard-finished terra-cotta, until to-day its use is more general than stone, appearing in entire fronts, as a bold-faced impersonation of solidity itself.

Fire-resisting Materials. — From what has been said regarding the excellence of terra-cotta, it must not be understood that this material constitutes the *only* satisfactory fireproofing medium, but it is undoubtedly the best (when of the porous variety) if used with discrimination and intelligence. Terra-cotta has long been recognized as the standard to which other materials or systems have been compared, and while its satisfactory qualities have never been surpassed, other materials have been introduced from time to time as competitors in fire-resistance, particularly on the basis of price.

Brick, which is also a clay product, is probably the only material which may be considered as equal to the best grades of terra-cotta, and many conflagrations have amply demonstrated the fire-resisting qualities of good brickwork. But of other constructive materials, concrete alone has stood the test of repeated trials, regardless of the most determined opposition from many quarters. Practically all kinds of stone and stone masonry are wholly unreliable under fire- and water-tests, but concrete is now generally recognized as an entirely acceptable fireproofing medium.

But even the most enduring materials, used with the greatest discrimination, are limited as to their effectiveness and resistance, and the duration and degree of exposure must,

therefore, be kept within reasonable limits. The best that can be done is to reduce the inflammable elements to a minimum, and endeavor to confine the fire by means of fireproof floors and partitions, so that it may do no injury beyond the consumption of local woodwork and furnishings.

Fireproofing Requirements.—With this general review of the fire problem, it is evident that a fireproof structure must possess:

1. General excellence of design.
2. All floors of fireproof construction.
3. All columns of masonry or steel, protected from fire.
4. All outside piers and walls of masonry or steel, protected from fire.
5. All partitions and furring of fireproof construction.

There are three methods of general design advocated at the present time as means of reducing the fire risk—the "slow-burning construction," the so-called "mill construction," and the still more effectual "fireproof construction."

Slow-burning Construction.—The term slow-burning construction is applied to buildings in which the structural members, carrying the floor- and roof-loads, are made of combustible material, but protected throughout from injury by fire, by means of coverings of incombustible, non-heat-conducting materials. Thus the wooden floor-joists are protected on the under side by a single covering of plaster on metal lath, while a thickness of $1\frac{1}{2}$ ins. of mortar or incombustible deadening is required above the joists. Columns, if of oak, with a sectional area of 100 sq. ins. or over, need not have special fireproof coverings. Partitions and elevator enclosures must be wholly of incombustible material, and no wood furring is allowed.

Mill Construction.—Buildings of mill construction are those in which all floor- and roof-joists and girders have a sectional area of at least 72 sq. ins., with a solid timber flooring not less than $3\frac{3}{4}$ ins. in thickness. Columns of wood need not

be protected, but they should have a sectional area of at least 100 sq. ins. Partitions and elevator enclosures are of incombustible material, and no wooden furring or lathing is used.

Fireproof construction has already been defined. The two types first mentioned do not, then, depend on the use of materials wholly incombustible, but rather on the judicious design and careful use of ordinary building materials, the aim being to provide structures so open and free from fire-lurking corners that they may offer no obstacles to a speedy suppression of the flames. These types are peculiarly adapted to large mills, warehouses, and the like.

Fire-resisting Design.—The scientific fireproofing of a building does not consist in a proper selection of materials alone, for a structure may be reasonably secure against accidental fire, or the extension of fire, even when built of combustible materials; nor does it lie merely in guarding against the causes of fire. It can be secured only by a thorough acquaintance with all the general features and minutest details of all kinds of structures, and by a quick perception "for the numerous elements of danger that are constantly creeping into modern systems of buildings." The plan must be carefully studied to secure means of cutting off communication between floor and floor, and between and around dangerous sources, isolating, if possible, all stairways and elevator-shafts by means of fire-resisting walls, and confining all power and mechanical plants in such a way that there can be no possible means of fire extension. It is true that most high office buildings do not possess the isolated stair-well or elevator-shaft; but if they do not, great care should be taken in making the halls and corridors of more than ordinary security. They will still be the means for a rapid distribution of smoke from floor to floor, and thus make the danger from suffocation assume an importance equal to that of fire.

No less important is the cutting off of all communication

between pipe- and air-passages. Piping and passages of all kinds should be carefully considered as a part of the fundamental design, for they not only become great eyesores from their exposed positions in offices, but they also serve to make many of our fireproofing endeavors quite useless.

The architect or engineer must finally be well informed in regard to the details and varied uses of approved fireproofing materials. These must include terra-cotta in all the different shapes made by the terra-cotta companies, cement, concrete, fire-brick, asbestos, mackolite, etc. A judicious and economic use of all these materials is necessary, so that the most practicable form may be chosen to secure the desired end.

Some of these important minutiæ may properly receive detailed attention, when we remember that the strength of a structure is gauged by its weakest point.

The metal columns, for example, are properly figured for their safe dimensions, but from this step on they are apt to become a bugbear to both architect and owner, the former desiring to reduce their size to a minimum on account of appearance, while the latter considers that they deprive him of the revenue of just so much floor space. Any measures are therefore adopted to reduce their size. First, the various waste-, heat-, and supply-pipes are run up alongside the columns from floor to floor. For the passage of these pipes openings must be made in the tile floor arches, which, in the rush of building operations, may never be properly filled up again. These openings come *inside* the line of the fireproof slabs of the column, thus forming one long continuous flue from basement to roof. The finished line of the fireproofed and plastered column is often not more than 2 ins. from the extreme points of the metal-work, and then, deducting $\frac{1}{2}$ in. or $\frac{3}{4}$ in. for plaster, little enough is left for the fireproofing proper. The various pipes before mentioned will very often project even farther than the column itself, thereby tempting

the fireproofer to trim and shave till the original *little* has become still *less*.

Lessons from Past Fires.—In the Athletic Club Building fire some of these points were illustrated with glaring prominence. A steel framework and fireproof covering having been used as the main elements of construction, further consideration of fire hazards were apparently slighted. In no case did the fireproofing extend more than 2 ins. from the outermost edge of the ironwork, while *wooden* nailing-strips were embedded in the tile at intervals of about 3 ft. starting from the floor (a 4-in. face exposed), making successively 3 ft. of tile and 4 ins. of wood. These nailing-strips were employed as grounds for the panelled oak wainscoting, and a further error was made in leaving an air-space behind this panelling, with no "back" plastering. The ceiling also left an air-space, due to 1-in. raised nailing-strips.

As a matter of course the wooden grounds around the column burned out, letting the fireproofing fall in 3-ft. sections. It so happened that but two columns were badly bent by the intense heat, but who can say what the stability of those re-used unbent columns really is? Were they cooled slowly, or suddenly by the application of streams of water, and thus rendered brittle, and were they heated unevenly, thus causing great strain in the material on but one side of the column? What was the amount of expansion and contraction? No experiments could be made with reasonable economy and safety to satisfy these queries, leaving the present state of the building an uncertain conjecture.

Fireproof Partitions.—Both the Pittsburg fire and the Home Insurance Building fire demonstrated the necessity for better partition construction, and the unreliability of plaster and metallic lath substitutions for terra-cotta blocks. In the Horne office building, wooden nailing-strips used for the attachment of the base-boards were responsible for the resetting

of nearly all the partitions. Aside from this defect the partitions were nearly as good after the fire as before. In the Home Insurance fire, many of the terra-cotta partitions were weakened or wrecked completely through being built *upon* the wood flooring, while the extended use of wooden door- and window-frames in such partitions was also responsible for a great deal of damage.

Fireproof Doors and Windows.—Again, these two fires clearly showed the necessity for protecting exposed window areas against possible external attack by fire, and this danger, as well as the objection to wood doors and window-frames, etc., in fire-resisting partitions, may be met through the use of a system which is now largely growing in favor. Doors, door-frames, window-frames, and sash are now made with a wood body or core, covered with sheet steel or copper which is hydraulically pressed to give the proper form to the panels, mouldings, etc. For exterior use, pure sheet copper is preferably used, of bright or green acid finish, while for interior use in partitions, etc., plain sheet steel is employed, ready for painting or graining. Sheet bronze, brass, or electro-plated metal may also be obtained. Fire-tests have proved the doors to be of admirable fire-resisting qualities, while the window-frames and sash, combined with wire glass, form a most admirable protection against external hazard. A building built in Boston, 1898, for the New England Telephone and Telegraph Co., has every exterior window and door of this character, the windows being glazed throughout with wire glass.

Installation of Piping, etc.—The proper installation and distribution of the mechanical features in a modern office building have been given considerable attention by John M. Carrére (see *Engineering Magazine*, October, 1892), and the system proposed by him will undoubtedly add greatly to the efficiency of fireproofing, and remedy many of the weak details just con-

sidered. In order to avoid chases, or continuous flues, the lowering of the hall ceilings is suggested, "thereby obtaining a horizontal space under the floors of the halls at each story, lined and fireproofed, where all the mechanical features except steam heat can be placed" (see Fig. 7). An arrangement of this

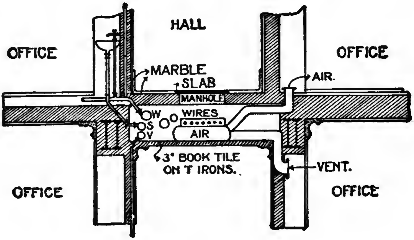


FIG. 7.—Arrangement for Pipe-space in Corridors.

character would certainly possess many great advantages—it would always be accessible for repairs, easy of connection with all offices, and would serve as a *safe* and at the same time hidden conduit for all wiring, piping, and ventilating air-ducts, either exhaust or indriven. The additional expense would not be great either, and when its permanency is considered, never being affected by the moving of partitions, etc., as is now the case, it is surprising that such a system has not attained more general use.

At the ends of these horizontal ducts are vertical chases or ducts built solidly of fireproof blocks or brick from cellar to roof, and connected at each floor with the horizontal leads, but still partitioned off at every story with wire and plaster partitions, to prevent the spread of possible fire. All of the vertical risers could be placed in these chases, thus avoiding the unsightliness of pipes in the office space, or the necessity of placing such piping within the column space.

CHAPTER III.

TYPICAL BUILDINGS—ERECTION, PERMANENCY, ETC.

MANY of the details which will be discussed in the following pages may be better appreciated in their relation to the whole subject if a few typical skeleton structures are examined. The scope of this outline will not permit of a discussion of the architectural problems involved in the design of a modern office building, hotel, or any of the structures which are now built according to skeleton methods. The points here considered are, rather, those of construction pure and simple. But the comprehensive view of the subject necessary to the architect or architectural engineer may only be obtained through an accurate knowledge of the manifold items which become parts of a successful plan. These accessories to the mere framework lie within the province of the engineer as well as of the architect, and here, as in the execution of the external expression of architectural engineering, a perfect harmony must exist between the two branches in the perfection of all mechanical details, if results are to be secured which may be looked upon as creditable to both professions.

The value of such accessories may be more fully realized when the self-sufficiency of a typical office building, containing all modern improvements, is considered. Electric light, the telephone, mail-chutes, and well-appointed toilet-rooms are already demanded as absolute necessities, while many examples provide telegraph and messenger service, cigar- and news-

stands and barber-shops, besides restaurants and cafés in the basements. It is true that many of these factors would seem to have little bearing on the duties of the engineer, and yet it was just such conditions, imposed on the designer of the foundations of office buildings, that produced the successful development of the so-called raft or floating foundations, in order that the basements might be unencumbered by the large

FIG. 8.—Chicago Stock Exchange Building. Adler & Sullivan, Architects.

pyramidal masses of stone previously used as footings, and the basement space might be added to the available renting area, or be used for the mechanical plants. The rigid economy of floor space which is demanded may only be obtained by careful attention to the most advantageous uses to which the different floors and rooms in the structure may be put.

Typical Office Buildings.—Some examples of typical office buildings in various cities will now be given, as illustrating prominent types of veneer construction.

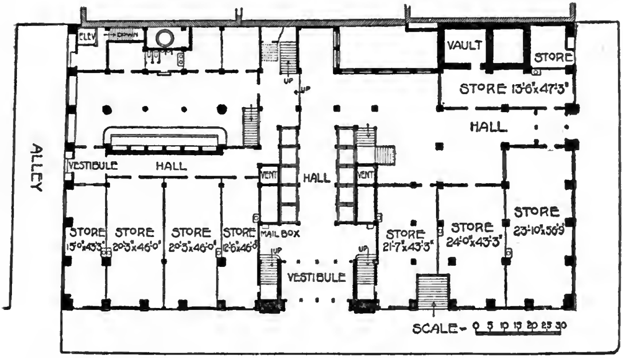


Fig. 9.—Chicago Stock Exchange Building. Ground-floor Plan.

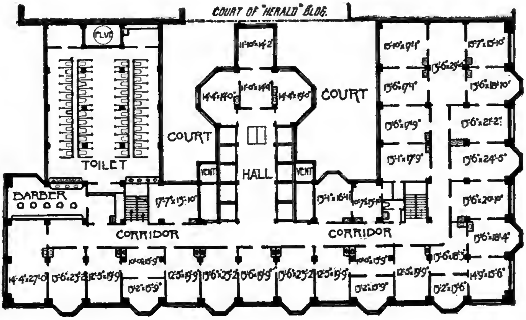


Fig. 10.—Typical Office-floor Plan, Stock Exchange Building.

The Chicago Stock Exchange Building is illustrated in Fig. 8. The entire façades are constructed of a yellow-drab terra-cotta, the lower stories and the main cornice being richly modelled in intricate ornamentation peculiar to the work of these architects. The interior court is faced with white enamelled brick.

Fig. 9 is a plan of the ground or street floor, showing the entrance vestibules, elevators, café, and store areas.

Fig. 10 shows the arrangement of the offices, etc., on the sixth floor. The toilet-rooms, barber-shop, vent spaces, and the arrangement of the lighting courts are plainly shown.

The Marquette Office Building, Chicago, is shown in Fig. 11. The exterior walls are built mainly of dark-red brick, with terra-cotta cornice and trimmings. A spandrel section at one of the upper floors is given in Chapter VI.

Fig. 12 illustrates one of the typical floor plans, with possible sub-divisions of large office areas. Many of the floors in the larger office buildings are never sub-divided until rented, in order that the arrangement of the partitions may be made to suit the tenants.

Fig. 13 is the Reliance Building, Chicago, the typical floor plan being as shown in Fig. 14. This arrangement of offices is intended for rooms to be used in suites. The pipe space at the side of the elevators, and the space-for counterweights behind the elevators are plainly shown, as is the circular smoke-flue.

The elevator accommodations in these various buildings may be seen on the plans. Rapid passenger and freight service must both be provided for, and the necessary space allowed for the hydraulic cylinders in the basement, as well as for the vertical counterweights. Beams must be supplied to support the elevator sheaves, and water-tanks located to supply the hydraulic cylinders.

If the basement lies below the street or sewer level, and it is to be occupied by stores, cafés, or by the boiler- and engine-

FIG. 11.—Marquette Building, Chicago. Holabird & Roche, Architects.

rooms, an ejector-pit will be necessary to raise the sewage to the proper level. Pumps for water-supply, dynamos for electric light, boilers and steam plant for power and heating—all must be definitely determined and carefully weighed in their relation to the character of the building, and as affecting the design of foundations and all structural details.

The following data may be of interest as descriptive of some of the mechanical furnishings of one of Chicago's most celebrated office buildings, the Masonic Temple, shown in Fig. 15:

The entire drainage is carried through the building by means of a system of vertical risers, about one-half of which connect directly with the street mains, through piping suspended from the basement ceiling. The remainder of the risers, and all drainage from the boiler-room and basement space, are connected by a system of underground piping with two 50-gallon Shone ejectors, placed in a pit in the basement, from which the sewage is forced to the street sewer. This was necessary in order to keep the basement stores, cafés, etc., free from exposed pipes. All vertical pipes in the building, both for water-supply and drainage, are carried in fireproof pipe-spaces especially provided. The water-supply is pumped from the city mains by pumps located in the basement, to storage tanks on the twentieth floor, with a combined capacity of 7,000 gallons. On the twentieth floor also are four compression elevator tanks of 18,500 gallons capacity total. For elevator and water-supply service seven pumps are required, having a total capacity of from 2,000 to 3,800 gallons per minute.

Each office and store has a private wash-basin, with general toilet-rooms and barber-shop on the nineteenth floor. The main toilet-room contains 64 closets, besides additional rooms on the third and twelfth floors and in the basement, with from 8 to 18 closets each.

Forty thousand square feet of radiation surface are required,

all in direct radiation. The steam is supplied on the "overhead" system through 16-in. mains running directly to the attic, thence around the exterior walls and down. Six dynamos supply 7,000 16-candle-power lamps. For the power and steam plant, eight horizontal tubular boilers are used, with a total of 1,000 horse-power.

There are several features in the Masonic Temple design worthy of especial note. Several of the upper floors are

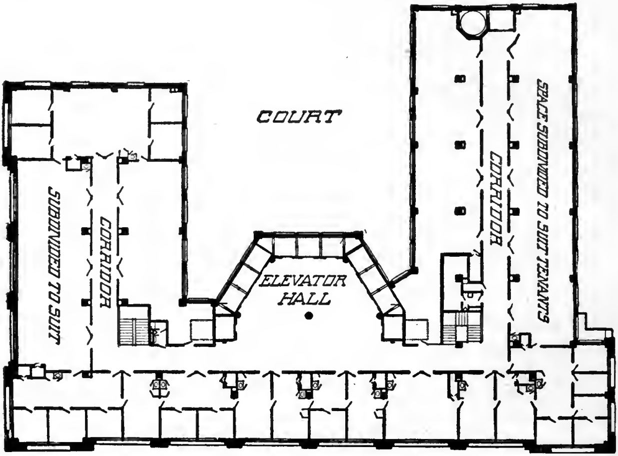


FIG. 12.—Marquette Building, Chicago. Typical Office-floor Plan.

devoted to Masonic purposes, and the large assembly-, drill-, and banquet-rooms were kept free from columns by spanning the areas with lattice girders, on which rest the arched ceiling and roof-trusses. The interior court also possesses a special feature, viz.: galleries provided at each story for the lower ten floors. This plan was intended to attract small storekeepers and the like as occupants of the adjoining stores or offices, thus concentrating many tradesmen under one roof. The scheme has not proved a success.

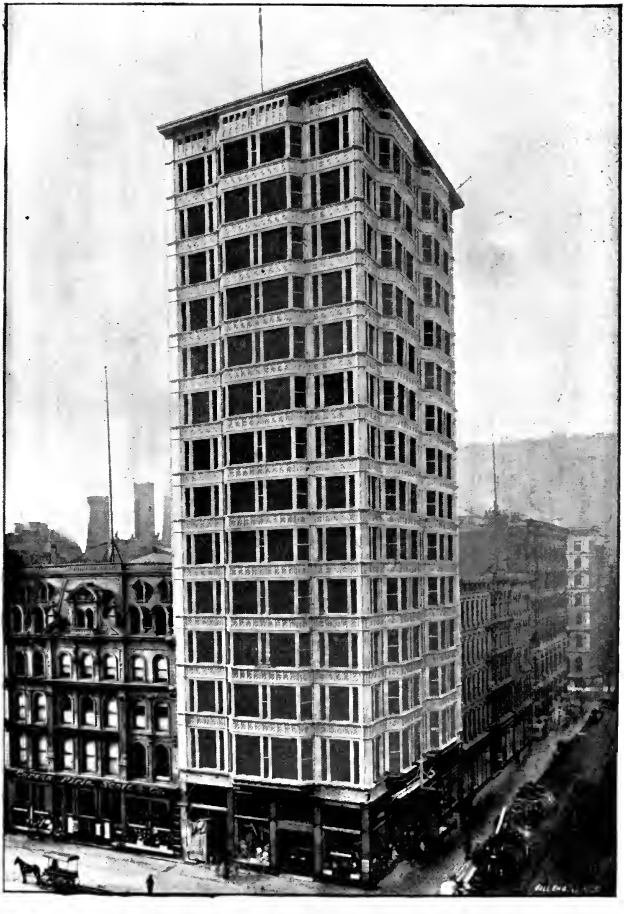


FIG. 13.—Reliance Building, Chicago. D. H. Burnham & Co., Architects.

The roof of the Masonic Temple is covered by an enclosure of glass, serving as a summer-garden and place of observation.

A perspective of the New York Life Insurance Building,

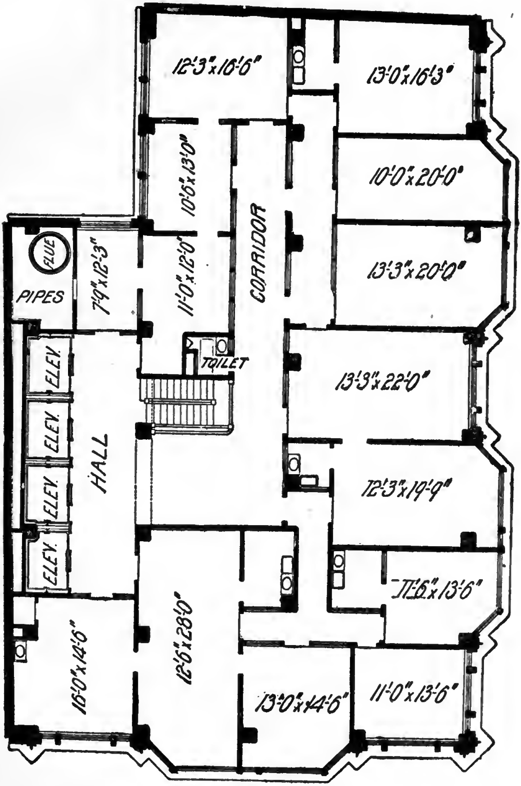


FIG. 14. Reliance Building, Chicago. Typical Office-floor Plan. (For framing-plan, see Fig. 59.)

Chicago, is illustrated in Fig. 17. The lower three stories are built of granite, with brick and terra-cotta above. The plan of the first floor, devoted to banking purposes, is shown in

Fig. 18, while the typical office-floor plan is given in Fig. 19.

Fig. 20 is a photograph of the Park Row Building, New York City (R. H. Robertson, architect). A skeleton elevation of the side wall shown in this illustration is given in Fig. 165.

The Park Row Building is the highest office building ever erected, and it is very doubtful whether it will be found either desirable or profitable to erect other buildings as high as this one. This building was built in 1897–98, and a number of the constructive details are given in other chapters. The height includes 26 stories from curb to main roof, or 33 stories from the foundation to the extreme portion of the accessible interior. The height from the street-level to the base of flag-staff, which is the highest accessible portion of the building, is 390 ft. 9 ins., or from head of piles to base of flagstaff equals 424 ft. 6 ins. The building is equipped with nine electric passenger elevators running from the basement to the twenty-sixth floor, besides which two other elevators, one in each tower, run from the twenty-sixth to the twenty-ninth floor.

A photograph of the Broadway Chambers (Mr. Cass Gilbert, architect) is given in Fig 21. The lower three stories are built of granite, the main shaft is of brick, while the upper three stories are constructed entirely of terra-cotta. The color-effect in this building is as successful as it is unusual.

Figs. 35 and 36 show this structure in process of erection.

Figs. 22, 23, and 24 show the basement, ground-floor, and typical-floor plans respectively.

The Jewelers' Building, Boston, Mass. (Winslow & Wetherell, architects), is shown in Fig. 25. The lower two stories are of cast-iron, with buff-colored terra-cotta above.

Fig. 26 is of the Fort Dearborn Building, Chicago. A typical office-floor plan is shown in Fig. 27, while the typical framing plan is given in Fig. 58, Chapter IV. A number of spandrel-sections for this building are given in Chapter VI.

FIG. 15.—The Masonic Temple, Chicago. Burnham & Root, Architects.

New Half,
Veneer Construction.

Old Half,
Solid Masonry
Walls.

FIG. 16.—The Monadnock Building, Chicago.

The Gillender Building, New York City (Berg & Clark, architects), is shown in Fig. 28. This example constitutes about the extreme of great height compared to narrow width.

FIG. 17.—New York Life Insurance Building, Chicago. Jenney & Mundie, Architects.

A framing plan is given in Fig. 60, Chapter IV, and spandrel-sections and bay-window details are illustrated in Chapter VI.

Fig. 29 shows a typical office-floor plan of the Champlain Building, Chicago (Holabird & Roche, architects).

Fig. 30 gives a perspective of the Old Colony Building, by the same architects.

Fig. 31 is a photograph of the new building erected for Montgomery, Ward & Co., Chicago (Richard E. Schmidt, architect).

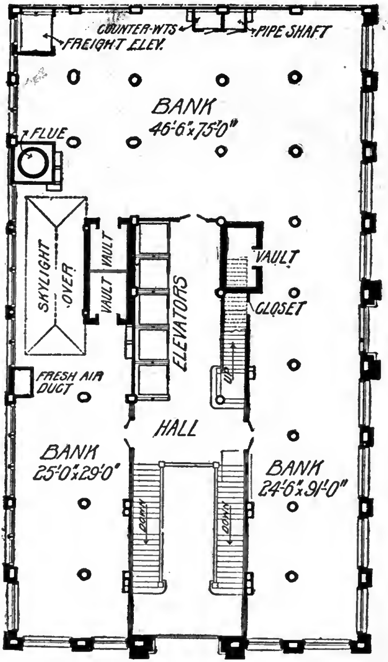


FIG. 18.—New York Life Insurance Building, Chicago. Plan of Banking Floor.

Fig. 32 shows the main entrance hall to the New York Life Insurance Building, Chicago, in which the walls, ceiling, and stairs are finished in Italian marble with mosaic floor. In many buildings the richness of the first story is further increased through the use of solid bronze for the elevator

grilles, stairs, transom- or door-grilles, directory-frames, and lamps.

The foregoing illustrations will serve to show the architectural treatment employed in representative office buildings,

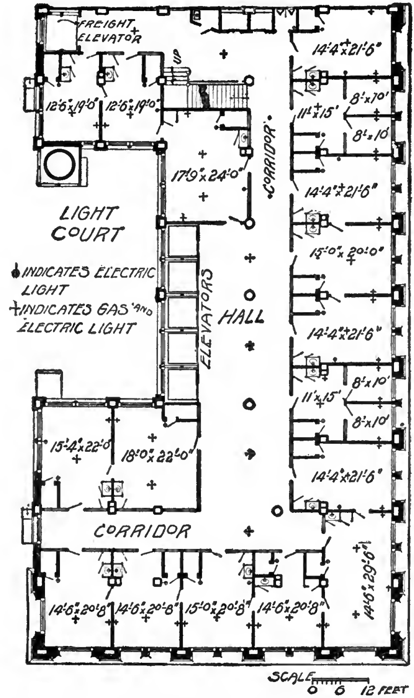


FIG. 19.—New York Life Insurance Building, Chicago. Typical Office-floor Plan.

while the floor-plans indicate the general arrangement of offices, halls, and entrances, besides the minor details of plan, thus making the conditions which determine the general features of construction apparent, in so far as the plan may affect the conditions of design.

Erection.—In skeleton or cage construction buildings, present demands as to the rapidity of construction make the method and apparatus for hoisting, handling, and assembling the various members, of great importance. Sharp competition, close estimates, and the demands of owners and architects regarding the speedy completion of contracts, serve to make the economy and rapidity of erection scarcely less important than economy and excellence in design. Many different systems of handling the steel frame have been adopted, and much special apparatus has been designed for this purpose, but the methods employed vary so much with locality and contractor, that no very general practice can be classed as standard. Simple gin-poles, single derricks or combinations of derricks, towers, steam-cranes, and elaborate travellers have been used under their own peculiar conditions; but that system will generally be found most advantageous which either facilitates the moving of the plant itself, or which renders much moving unnecessary. Any saving in shifting, anchorage, or guying, tends to reduce the time employed, and hence the labor and expense.

Old-fashioned gin-poles and single derricks are still employed on small work, but on buildings of considerable size, some form of tower or traveling-derrick is generally used. Special steam-cranes, built for the purpose, have been used in some cases, these being operated on tracks which were quickly laid over the floor system. Such cranes would pull themselves up an incline, from story to story, as fast as erected. The crane-boom and engine-platform revolve on a pivot, so that the steel beams or columns require very little handling.

When the building floor-plan is of such dimensions that a derrick or traveler may move from end to end of the building, and at the same time reach out on either hand to the side walls (or even when only a portion of the width can be handled), a tower-derrick will be found advantageous, providing other

FIG. 20.—Park Row Building, New York. R. H. Robertson, Architect.

FIG. 21.—"Broadway Chambers" Building, New York City. Cass Gilbert, Architect.

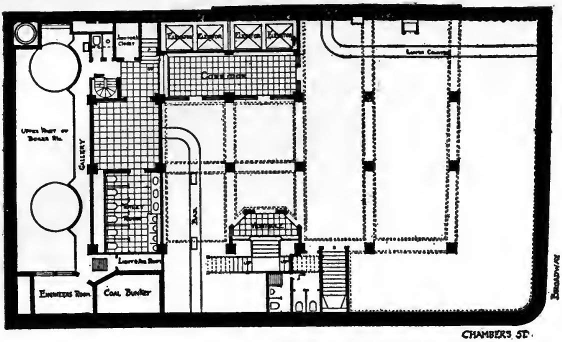


FIG. 22.—Broadway Chambers, New York City. Basement-floor Plan.

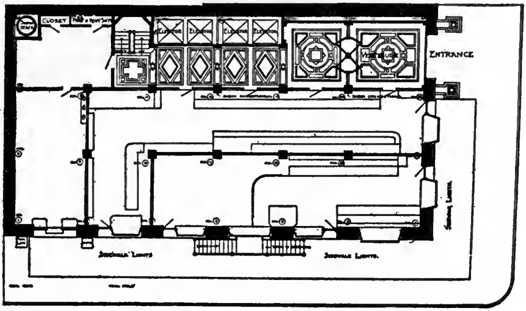


FIG. 23.—Broadway Chambers, New York City. Ground-floor Plan.

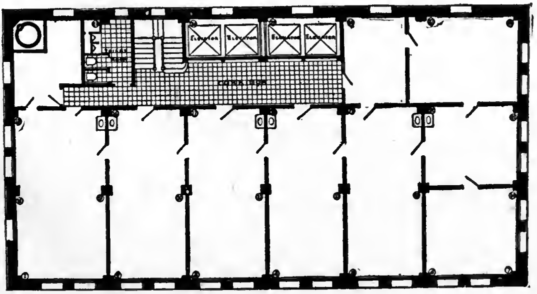


FIG. 24.—Broadway Chambers, New York City. Typical-floor Plan.

conditions are suitable for the use of a group of central booms.

Such a traveling-derrick or tower-derrick, which has been used with good results on skeleton buildings in New York, may be briefly described as follows: The derrick consists of a rectangular tower, about 24 ft. long, 24 ft. high, and 12 ft. wide, made of a horizontal rectangular steel framework at the bottom, which supports four wooden corner-posts. These corner verticals are connected at the top by horizontal timbers, running from post to post, and also by transverse struts placed about half-way up the tower. Diagonal rods with sleeve-nuts and pin-ends are placed in each of the vertical planes of the tower, also in the top and bottom horizontal frames. The vertical corner-posts are so arranged as to set back somewhat from the ends of the bottom iron frame, which projects at each end sufficient to receive the boom-seats, one at each corner. All joints are connected by steel cover-plates and bolts, the whole being arranged with a view to rapidity in erection or removal. The floor of the tower is supported on transverse I-beams which rest on the bottom frame, and on these beams planking is placed to receive the engine, besides the necessary coal- and water-supplies. The traveler is run on rails, spaced about 12 ft. apart, placed on loose flooring about 3 ins. thick, laid from beam to beam. To further facilitate the handling of material the moving derrick is often supplemented by a distributing-car which runs on a narrow-gauge track of light rails. The traveler, including engine and all, is easily raised from floor to floor by hand, by means of four breast-derricks.

This type of traveler was employed on the Commercial Cable Building, New York, and on the Siegel-Cooper Building, where seven complete tiers, aggregating between 7,000 and 8,000 tons, were erected in nine weeks actual working time. In this case two travelers were installed on opposite sides of the same floor, and a gang of twenty men with each derrick

Fig. 25.—Jewelers' Building, Boston, Mass. Winslow & Wetherell, Architects.

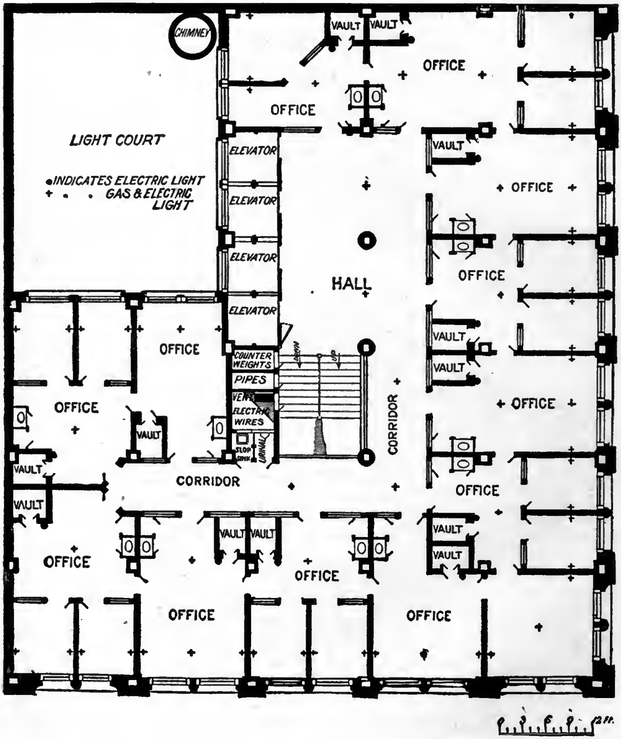
would erect about twenty bays of ironwork in a day, each bay being about 24 ft. square.

The rapidity of erection is not proportional to either the

FIG. 26.—Fort Dearborn Building, Chicago. Jenney & Mundie, Architects.

cubical contents of ordinary buildings or to the linear height, as an average rate of setting steel frames may be placed at about two tiers of beams per week of six working-days of ten hours each. This rate is largely independent of the actual size

of the building, except in large areas where the material cannot be handled directly from the street to final position with one operation of the boom. In many cases of quick-time contracts,



FIG, 27. Fort Dearborn Building, Chicago. Typical Office-floor Plan.

this rate is often greatly bettered. In the Unity Building, Chicago, seventeen stories, the erection of the metal framework from basement columns to finished roof was accomplished in nine weeks. Cast columns were employed in this case. In

FIG. 28.—Gillender Building, New York City. Berg & Clark, Architects. American Surety Building in Background to left.

the Fisher Building, Chicago, 1895, the entire steel skeleton above the first floor was erected in twenty-six days, without overtime or night work. This included nineteen stories and an attic.

Figs. 33 and 34 show the progress made in the erection of the Reliance Building, Chicago, from July 16, 1894, to August 1, 1894.

Figs. 35 and 36 show the Broadway Chambers, New York, 1900, during construction. The eighteen stories of this steel

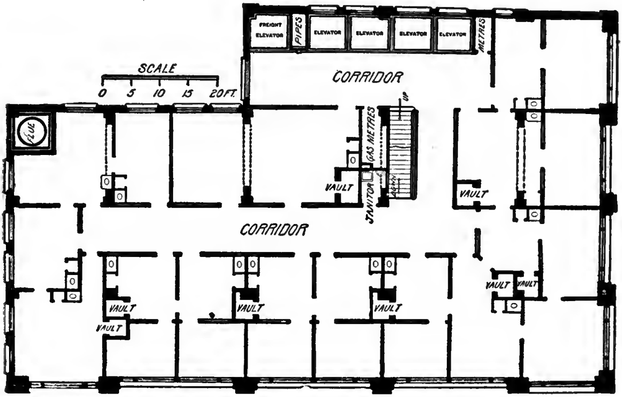


FIG. 29.—Champlain Building, Chicago. Typical Office-floor Plan.

frame, aggregating 2,000 tons, were erected complete between the dates Oct. 15 and Dec. 18, 1899.

For the successful erection of the frame, much depends upon an accurate alignment of the column bases. These should be carefully tested as to both position and level. The bases are either grouted with cement, or bolted to the foundations, but where cast column bases rest on masonry piers or

FIG. 30.—Old Colony Building, Chicago. Holabird & Roche, Architects.

FIG. 31.—Montgomery Ward & Co.'s Building, Chicago. Richard E. Schmidt, Architect.

footings, any considerable grouting is not advisable. The only grouting that should be permitted in tall buildings would be in leveling up the tops of the concrete footings to receive the masonry courses, or in a very thin layer between the

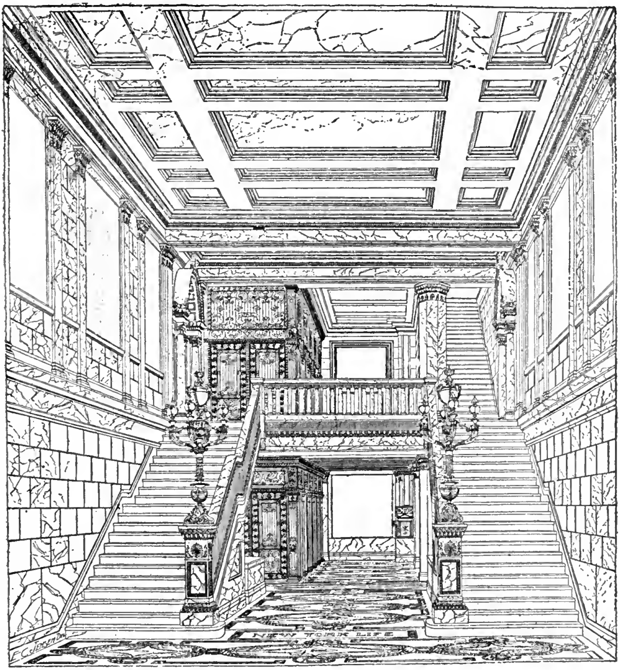


FIG. 32.—Entrance Hall, New York Life Insurance Building, Chicago.

column pedestal and the masonry bed. The cap-stones should always be brought to the most accurate bed possible, with grouting used as a thin cement and not as a leveler. Accurate re-dressing of the cap-stones after setting is much to be preferred.

All riveting and punching of the steel members is done at

the shop, besides the usual coat of oil or paint. This leaves only the assembling and field riveting to be done on the ground, including the adjustment of the laterals or wind-bracing, the placing of separators and tie-rods, and the field painting.

The columns are now generally made in two-story lengths, or occasionally in three-story lengths, and this practice aids much in saving time and expense in erection. The column splices are placed from 12 to 24 ins. above the floor-levels (see "Column Splices," Chapter VII), so that the floor-beams or girders may rest on brackets or shelf-angles near the tops of the columns, thus acting as braces during erection. In the splicing of columns, shims or wedges should never be permitted, as such practice leads to serious abuse in careless hands, and nails, pieces of slate, etc., are often used by the men to secure proper adjustment. The work should be made true and perfect through the accurate planing or "facing" of all contact bearing-surfaces, the facing of column ends always being done at exact right angles to the column axis.

Beams and girders are first bolted temporarily in place, about one-third of the holes being filled. The riveting gang then follows behind the erectors, making permanent connections with iron rivets heated in portable forges. Field riveting has now entirely superseded the use of bolts in skeleton or cage construction, or indeed in any character of high-class building work. Bolted connections were tried, but were soon discarded on account of the cracks which developed in the plastered ceilings. These cracks were always found to radiate from the column connections with the floor system, thus demonstrating the play of the bolts in the holes. A list of the required field rivets is made in the shop, including an excess of from 5 to 25 per cent. of the actual number required. This percentage is added for waste, loss, and the burning of rivets in the field. A greater percentage should be added for short

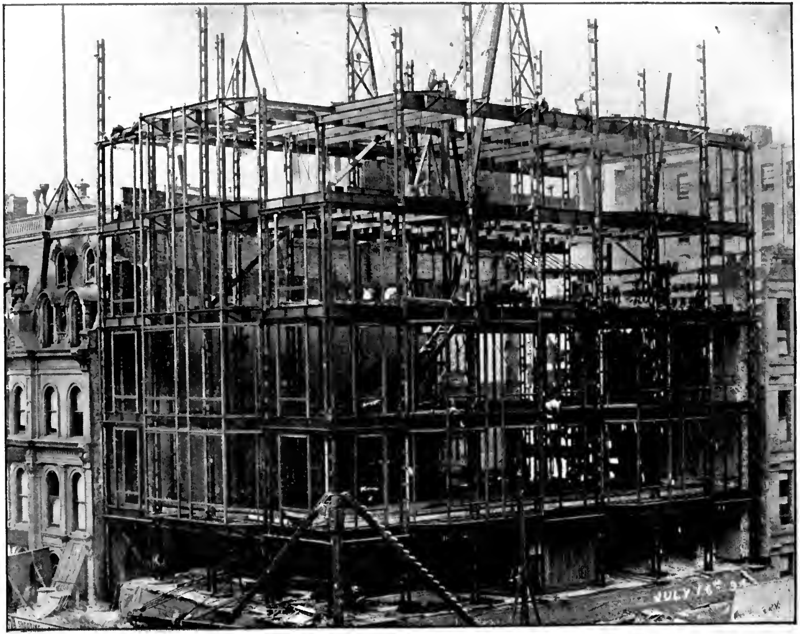


FIG. 33.—Reliance Building, during Construction.
July 16, 1894.

FIG. 34.—Reliance Building, during Construction.
Aug. 1, 1894.

FIG. 35.—Broadway Chambers, during Construction.
Nov. 9, 1899.

FIG. 36.—Broadway Chambers, during Construction.
Dec. 21, 1899.

rivets than for long ones, as long rivets may be cut down to make shorter lengths. A riveting gang of five men will average about 200 rivets a day of nine hours, under good conditions. This makes a cost of about 7 to 8 cents per rivet.

After erection, the steelwork should receive one or two coats of paint. If the cost need not be too carefully considered, two coats in the field are to be recommended, in which case the first and second coats should be specified of different colors. This enables one to see at a glance that the second coat has not been skimmed or slighted, as will often be found to be the case unless given very careful inspection. For one coat of red-lead paint, one gallon may be allowed to about two tons of average weight structural steelwork.

Rapidity of Erection.—The skeleton or "veneer" type of construction possesses great advantages in economy of time required for erection, as work can be pushed on the walls at different stories at one and the same time. Thus on the Manhattan Building, Chicago, the main cornice of terra-cotta was completed before the wall was built up beneath it. On the Unity Building the granite base-wall was being built at the first and second stories, the pressed-brick face was being placed at the twelfth-floor level, while the hollow-tile arches were being set for the fifteenth floor,—all at the same time.

The rapid progress made in the erection of the New York Life Building, Chicago, is shown by the following:

July 17. Old building torn down to grade.

July 31. Laid out new footings.

August 17. Started setting basement columns.

August 31. Started laying granite.

September 5. Started setting tile arches.

September 18. Started laying terra-cotta facing.

September 29. All steel set.

November 9. Tile floors all set.

November 11. Terra-cotta all set.

November 12. Started plaster.

December 2. Steam plant completed—turned steam on in building.

Of the 671 individual columns in this building, but a single one required "shimming." A thin steel wedged plate was used, forged to fit. The columns were tested for alignment at frequent intervals. An average of twenty-five working hours was required to set the steelwork for a complete story.

The following dates will serve to show the time required in the erection of one of the latest New York office buildings, viz., the eighteen-story Atlantic Building, corner of Wall and William streets (Clinton & Russell, architects):

May 9, 1900. Tearing down started.

June 15, 1900. Caisson foundations started.

September 1, 1900. Steel frame started.

October 8, 1900. Brickwork started on street fronts.

December 10, 1900. Building topped out.

January 1, 1901. Steam turned on.

January 22, 1901. First hydraulic elevator started.

March 1, 1901. First offices ready for tenants.

Permanency of Skeleton Construction.—Aside from the question of fire resistance, much discussion has arisen from time to time as to the permanency of skeleton construction. This controversy between friends and indifferent observers of skeleton methods was also aggravated by the reluctance of the supervising architect of the Treasury seriously to consider such construction as worthy the dignity and solidity of government edifices—notably in the new Post-office Building for Chicago. While the architectural pros and cons of terra-cotta and steel, or concrete and steel, versus solid masonry construction may not here be discussed, the engineering side of this matter becomes one of great importance. Serious as it is, it must still be admitted that it depends largely on personal views, for the want of reliable data under present conditions. Many

architects are not slow to pronounce judgment against such practice, while others warmly champion the cause of steel in combination with tile, concrete, or cement. This divergence of opinion was well shown in an interesting discussion before the American Institute of Architects on this very subject, where examples of the deterioration of iron or steel under peculiar conditions were emphatically offset by instances of remarkable preservation under other peculiar conditions. The point would then seem to be to *define* these conditions. Prominent architects, engineers, and builders have said that experience seems to show that, if *no* lime mortar is used, the corrosion of the metal will not amount to enough to be of any danger; while others point to the well-known *preservative* qualities of lime, and urge its exclusive use in connection with iron or steel. Our knowledge of wrought-iron or steel, therefore, under definite variations of heat and moisture, and in association with limes, cements, and concrete, as found in present practice, must continue to be unsatisfactory until defined by more accurate data. American engineers and builders show their daily faith in such combinations of material, and this type of construction is rapidly becoming more and more general in the United States.

The effects of lime, whether as one of the ingredients of mortar or of limestone, as a corrosive factor in connection with ironwork, seem to depend very largely upon the peculiar conditions of each particular case. Examples are recorded of anchorage cables in American suspension bridges which were found, on disclosure after some years, to be partly eaten away where the strands had come into permanent contact with the limestone masonry. The presence of water was possibly accountable for this corrosive action; but it becomes a very difficult matter to construct masonry which will allow of no permeation of moisture, especially in walls, piers, or foundations, as found in building practice. Dry air and pure water

produce but slight oxidizing effects on iron or steel; "but when the former becomes moist, and the latter impure or acidulated, oxidation of the material is speedily set up, and, when once commenced, unless the process is arrested, its ultimate destruction becomes a simple question of time." The use of lime mortar would, therefore, seem limited to localities where no fear of moisture may be anticipated; for any dampness in combination with the lime must soon show its effects on the metal-work.

Considering the parts of a skeleton structure which are exposed to the weather, or liable to the presence of moisture, we have: all exterior walls, piers, etc., and the basement members, including foundations. From the foregoing it would seem that lime mortar should not be used in any of these positions. The foundations and basement walls, columns, etc., are either surrounded by constant moisture, or by wet clay or earth itself, while the exterior walls and supporting steelwork are subjected to the climatic changes, frost, rain, and penetrating dampness, which must sooner or later pierce the terra-cotta and brick envelope, and so reach the metal-work. For such positions cement mortar should undoubtedly be used; it seems a most perfect conservator of metal-work, and instances are recorded of iron found in perfect condition after a 400-years' entombment in cement concrete below water. Links of anchorages in American suspension bridges have been taken up after many years in a perfect state of preservation where embedded in cement. A further recommendation of the use of cement lies in the fact that the thermic expansion of Portland cement is practically the same as that of iron—a fact which insures perfect cohesion under any changes of temperature.

The interior members of the framework do not need as careful consideration, being maintained at a more uniform temperature, and protected from the exterior dampness. Interior columns, the floor system, and wind-bracing would, therefore,

seem safe in connection with lime mortar, but it is questionable whether the best work should not call for cement mortar and even cement plaster throughout. Cement has rapidly cheapened of late years, and cement plasters are largely being used on account of their better fire-resisting qualities.

It has been suggested to rely entirely on the preserving qualities of cement rather than on a proper painting of the metal-work. Prof. Bauschinger states that his experiments show a cohesion between iron and concrete, after hardening, of from 570 to 640 lbs. per square inch. This is even more than the tensile strength of the best concrete, but in building work a perfect union between the cement mortar and metal-work can never be attained at all points, and a thorough coating of paint must largely be relied upon.

In the surrounding of the metal framework by masonry or terra-cotta, it has been found, after an experience of fifteen years, that wherever masonry or terra-cotta shapes are so employed as entirely to cover the surfaces of the beams, girders, or columns with the cement mortar in which these coverings are laid, practically no oxidation takes place; while beams, girders, or columns which are simply protected, but which do not have the direct contact of the mortar with the steel, have frequently been found seriously oxidized.

In selecting materials for fireproofing purposes, their influence and action upon the life of the framework must not be neglected. Thus, while cinder-concrete is most enduring from a standpoint of fire resistance, more so than stone-concrete, still the employment of cinder-concrete in direct contact with steelwork is to be seriously questioned, due to the corrosion caused by the alkalies contained in the cinders.

Deterioration due to the leakage or radiation from supply-, waste-, or vent-pipes, must also be considered and provided against by keeping all such piping in ducts or chases *outside* of the fireproofing or protective coverings around the metal-work.

For more extended data as to permanency and corrosion, and the relative values of ordinary building materials when considered in relation to this subject, the reader is referred to the more complete discussion given in the author's "Fire-proofing of Steel Buildings."

Painting.—Excepting, therefore, such steel members as are completely surrounded by cement mortar, no more practicable method of protection is known than a good paint well applied, and the painting of the metal framework must thus constitute the principal safeguard against deterioration and corrosion, and, as the annual tonnage of steel shapes entering into building construction is increasing so rapidly, the importance of adequate protection is correspondingly increased.

The entire question of painting (including the condition or preparation of the steel or iron before paint or oil is applied, the kind of paint, the quality to be employed, and the best methods of application), is one of the utmost importance, and yet, in many particulars, of wide divergence in practice. For a more extended reference to this subject, several very interesting and valuable books and papers may be referred to,* a study of which will reveal great differences of opinion as regards materials and methods, and yet concurrence as to the principal considerations involved.

All agree that almost any attempt to prevent the deterioration or corrosion of metal-work by painting is of some benefit, and that, the more conscientious the effort, especially in the method of application rather than in the material, the more trustworthy will be the result.

To secure painting of permanent value, a clean scaleless and rustless surface is first necessary. Steel plates and shapes,

* See "Metallic Structures: Corrosion and Fouling, and their Prevention," J. Newman. "Painting of Iron Structures Exposed to Weather," and discussion, Trans. Am. Soc. C. E., vol. xxxiii. No. 6. M. P. Wood in Trans. Am. Soc. M. E., vol. xv.

when delivered from the rolls which form them to the cooling beds, are largely covered with scales, which, adhering only partially to the surface, offer the intervening cracks or joints as vulnerable points for rust. Almost at once after being rolled, structural steel is stored or handled out of doors for a varying period, both at the mill, and then again at the bridge shop before the fabrication is started. This period of open-air exposure allows the process of rust to start under the scales, and, "if the rust so covered up has not begun to pit the iron, the chances are it will never do any harm; but if it is already well developed and of some thickness, it will have enough oxidizing agents in its pores to develop more oxide, swell up, crack the paint, and the continuation is obvious." *

The first requirement, therefore, for efficient painting, lies in the careful removal of all mill-scale, rust, grease, or foreign substance, before even the priming coat is applied. And this initial condition is the most difficult to obtain of all the requirements for good painting, as, with present mill and shop methods, the cleaning of scale or rust is done only very superficially, if at all, and even if inspected the average inspector is satisfied with the mere uniform coloring of the surface. All authorities agree that the first step in the preservation of metal-work against deterioration or corrosion, is in obtaining absolute cleanness of metal before the application of paint or oil, but this result can only be obtained at increased initial cost of the metal, and through more rigid and conscientious inspection. The result would be well worth the added cost.

"Better results would be achieved in this direction if engineers in charge of important new work were to specify that the material shall go directly from the rolls to an adjoining closed shop or cleaning shed, where the scale is to be removed by light portable power-driven wire brushes or other suitable

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* See E. Gerber in Trans. Am. Soc. C. E., vol. xxxiii.

means, and the pieces are at once to be immersed in a bath of pure linseed oil. Then these are to be sent to riveting or other shops when dry enough to handle, and, when the work is complete, they are to be sent to an enclosed paint-shop, where a good coat of paint approved or specified by the engineer is to be given before shipment, ample time being allowed for drying." *

With present mill methods, the best that can be done is to secure the most careful cleaning practicable, after which a coat of oil is generally preferred, especially if the work is to receive two coats of paint in the field. Oil forms a transparent protective covering, thus leaving visible defects which might have escaped detection at the shop; it penetrates joints and surface cracks better than when mixed with pigment; it will not rub off as easily as paint, and it forms a better priming coat than either the new metal or dried paint. Pure boiled linseed oil is generally specified, because it dries more quickly than raw oil, and the latter remains sticky for a considerable time, gathering cinders and dirt in transportation which require cleaning before paint is applied. If thoroughly coated with pure linseed oil, steel members will not suffer by waiting several weeks or even months for the final coats of paint in the field.

The field painting should be done as soon as practicable after erection, and this leads to the question as to what constitutes a good paint. Present practice is pretty well confined to the use of oil paints, such as iron, lead, or other pigments ground and mixed with linseed oil or some substitute for linseed oil; coal-tar, or asphalt, or mixtures in which asphalt is the principal ingredient. Competent and disinterested authorities differ widely in their estimates as to the value of these coatings. While many engineers, chemists, and men of long practical experience recommend oxide of iron paint, others,

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* See Geo. A. Just in Trans. Am. Soc. C. E., vol. xxxiii.

equally qualified to advise, advocate the use of red lead, graphite, and carbon paints. The rivalry between oxide of iron and lead paints is of long standing, while graphite paints are of more recent introduction, and hence of more limited use. Patent paints, and compounds which have had but a very limited use, should not be seriously considered unless recommended by those qualified to judge as to the ingredients employed and the preservative qualities which could or would be attained.

With a careful initial cleaning, good inspection, and proper application, it is safe to assume that either oxide of iron, red lead, asphalt, or graphite paint will give good results, *provided the materials are of the best.* In the summary of the paper previously referred to, Mr. Gerber (see Trans., vol. xxxiii. p. 529) states as a conclusion based on his very extended investigation that "Iron oxide is far preferable, as the author sees the matter, aside from the question of cost, and in cost the advantage is certainly with it." Also: "If metal has been properly cleaned and paint properly applied, there need be no fear that any paint, composed of pure oil with a good pigment, will not protect the metal so long as the paint lasts."

In the discussion which follows the above-mentioned paper, many well-known engineers and men of large experience in structural metal-work advocate red lead, citing tests and experiences to substantiate their opinions. The government specifications for ironwork in the Congressional Library at Washington stated that "all work not bower-barffed must be given one coat of pure red lead paint before leaving the shop."

As to asphalt or carbon paints, the following opinions are quoted from a paper by Mr. M. P. Wood entitled, "Rustless Coatings for Iron and Steel." * Speaking of true asphalt paint, made from natural or Trinidad asphalt—not the artificial

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* See Trans. Am. Soc. M. E., vol. xv.

product of coal-tar distillation—he says: "Its toughness, and adhesiveness to all bodies, wooden, fibrous, as well as metallic, are remarkably persistent and durable. Its covering quality is also excellent, and for the exclusion of moisture and prevention of rust it has no superior, if any equal." As to lamp-black or carbon paints, he states that "Lamp-black as a carbon is practically unchangeable and indestructible under ordinary atmospheric conditions, and being itself of an oily and elastic nature, its combination with oil forms an elastic, close-clinging coating,—one of the best preservative paints known in the arts."

All authorities, however, insist on the use of *perfectly pure materials*, and as the oil is the principal preservative ingredient in paint, the quality of the oil is of the utmost importance. From its many good qualities, linseed oil stands preëminently at the head of the list, but "the number of non-drying oils of a vegetable character that are available for the adulteration of linseed oil are over thirty; the greater number of which are commercially cheaper than linseed." To these must also be added many other fish, animal, and mineral oils. These substitutes or adulterations are extensively used, but on drying or being exposed to the air they are sure to crack, thus greatly lessening the durability and value of the preservative coatings. Many methods are employed for detecting the adulteration of oils, the most common being by means of bringing the oil into contact with strong sulphuric acid. See Ure's "Dictionary of Arts, Manufactures, and Mines," vol. ii. p. 301. Oil or spirits of turpentine, or "turps", and benzine dryers should never be used.

Equal care is necessary to avoid oxide of iron paints, containing a large proportion of clay as adulteration, or red lead paints with chalk and lime. Fraud can generally be avoided by dealing directly with manufacturers of good standing, instead of buying from low and irresponsible bidders.

Finally, no painting should be allowed in freezing or stormy weather. Paint should be applied when the material to be painted is as free as possible from dampness, and it must be remembered that the more area a paint covers, the thinner the film is, and hence the less it is able to protect the metal. A good heavy coat is far preferable to a thin one, and the spreading qualities claimed by paint manufacturers for their products should be considerably discounted. The relative cost and covering capacity of the paints in most general use, may be tabulated about as follows, the prices varying somewhat according to market fluctuations. The prices given are for absolutely pure materials.

	Cost per Gallon.	Reputed Covering Capacity of 1 Gallon. Square Feet.	Cost of Paint per 100 sq. ft.*
Oxide of iron paint.	$1.30	600 to 700	22 to 19 cts.
Red lead	1.95	500 to 700	39 to 28 "
Superior graphite	1.10	600 to 800	18 to 14 "

* Light structural work will average about 250 sq. ft., and heavy structural work about 150 sq. ft. of surface per net ton of metal.

Building Laws.—The following are the requirements of the New York building law in regard to the protection of iron or steelwork against corrosion, etc.:

"All structural metal-work shall be cleaned of all scale, dirt, and rust, and be thoroughly coated with one coat of paint.

"Cast-iron columns shall not be painted until after inspection by the Department of Buildings.

"Where surfaces in riveted work come in contact, they shall be painted before assembling.

"After erection all work shall be painted at least one additional coat.

"All iron or steel used under water shall be enclosed with concrete."

The Chicago ordinance makes no mention of paint or coatings to prevent rust in the metal framework, except as specified for fireproofing purposes as follows: "In all cases the brick or hollow tile shall be bedded in mortar close up to the iron or steel members, and all joints shall be made full and solid."

The Boston law requires a protection from heat only, by means of brick, terra-cotta, or by three-fourths of an inch of plastering.

The requirements for the protection of metal-work in foundations are given in Chapter X.

CHAPTER IV.

FLOORS AND FLOOR FRAMING.

THE engineering or constructive problems involved in steel building construction must naturally start with the load-bearing floor system, for upon the floors and floor-loads depend the calculations of the columns and foundations. In skeleton or cage construction, the walls are not relied upon for load-carrying capacity, but are themselves carried by those members of the floor system which connect the exterior columns,—while provisions made for wind-bracing may most properly be treated as a portion of the column design.

Starting, then, with the floor areas, the first requisite is the choice of a satisfactory floor arch of terra-cotta, concrete, or other material, or combinations of materials—and here a wide choice is offered the architect or engineer. A preference must not be based on form or appearance alone, as fulfilling architectural requirements, nor upon strength only, as satisfactory to the engineer; but form or appearance, strength, and *fire-resisting qualities* must all be given due weight in an intelligent selection.

Brick and Corrugated-iron Arches.—The oldest so-called fireproof arches consisted of I-beams, placed about 5 ft. centres, with 4-in. brick arches turned between, then levelled up with concrete containing the nailing-strips for the wooden flooring. Corrugated-iron, sprung from flange to flange, was also used in place of the brickwork, and this latter type may

still be seen in some of the more substantial buildings of that epoch, which have survived to the present time. This construction was decidedly faulty, however, not alone in the weakness of the arch itself under the action of fire, but in the fact that the lower flanges of the supporting I-beams, and the entire cast columns then in use, were left exposed to view, and, what was much more serious, to the possibility of contact with fire.

These heavy and unsatisfactory types, shown in Figs. 37

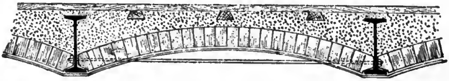

FIG. 37.—Brick Arch Construction.

FIG. 38.—Corrugated-iron Arch.

and 38, usually approximated 75 lbs. per sq. ft. dead load, for the arch and concrete filling alone.

Introduction of Terra-cotta Arches.—Present methods of terra-cotta floor arches practically resulted from the great Chicago fire in 1871. While this conflagration exhibited many admirable examples of fire-resisting brick and concrete arches, it plainly demonstrated the necessity for better methods, at reduced weight and cost. In 1872, therefore, flat hollow-tile arches were first patented and introduced in Chicago by Mr. Geo. H. Johnson, and at about the same time a similar but heavier construction was used in New York City in the corridors of the Post-office Building.

These early examples were naturally very crude as to workmanship and materials, but as terra-cotta arches proved to be light, substantial, and fire-resisting, their use soon became greatly extended, indeed almost universal in this country.

Early Forms of Tile or Terra-cotta Arches.—The earlier forms of tile arches were made as in Fig. 39, which shows the

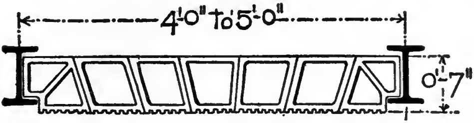

FIG. 39.—Terra-cotta Arch used in Equitable Building, Chicago (1872).

arch used in the Equitable Building in Chicago (1872), and Fig. 40, which shows tile arch in the Montauk Building,

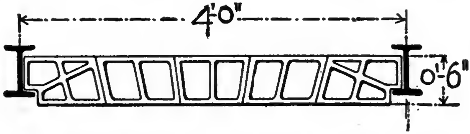

FIG. 40.—Terra-cotta Arch used in Montauk Building, Chicago (1881).

Chicago (1881). The latter may be said to have been the first building of modern design in Chicago. The arches were 6 ins. deep, with a span of 3 to 4 ft. But as these forms still left the lower flanges of the I-beams unprotected, they were soon superseded by the type shown in Fig. 41. This arch was used

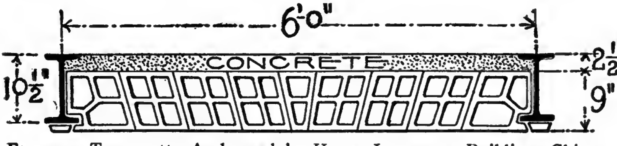

FIG. 41.—Terra-cotta Arch used in Home Insurance Building, Chicago (1884).

in the Home Insurance Building, Chicago (1884), the tile being 9 ins. deep and 6 ft. span. This was the first instance in which the beam soffits were protected against fire by anything more than plaster; and it is interesting to note that the introduction of soffit pieces, under the beam flanges, was due to an attempt to remedy the discoloration of the plastered ceilings,

rather than to improve the fire-resisting quality. Previous arch-blocks had been made to project about one-half inch below the bottoms of the beams, thus leaving recessed spaces under the beam flanges. These recesses were filled with mortar at the time of applying the first coat of ceiling plaster, but it was soon found that the cooler surfaces under the beams condensed the moisture in the atmosphere along these lines, and caused the soot or smoke from soft-coal fuel to accumulate, and to indicate the beams by black lines on the ceilings. This trouble first suggested the use of protection tiles for the beam flanges, a detail which greatly increased the fire-resisting qualities of terra-cotta arches.

Previous to the year 1883, the arch-blocks, excepting the skew-backs, had all been made without interior webs, but requirements as to strength and the increase of spans between the supporting beams soon caused the introduction of heavier and stronger types. In 1883, contracts for the floors in the Mutual Life Insurance Company's building, on Nassau Street in New York City, were awarded to a Chicago fireproofing company, and arch-blocks with both vertical and horizontal interior webs were employed. The arches weighed 33 lbs. per superficial foot, and were practically as shown in Fig. 41,—the arches used in the Home Insurance Building, Chicago, built at about the same time.

In the foregoing examples of arches, known generally as the "Pioneer" arches (because made by the Pioneer Fireproofing Company of Chicago), the voids in the tile blocks ran parallel to the supporting beams, and hence the principal or side webs of the individual tile blocks also ran parallel to the beams, or at right angles to the line of thrust in the arch. This limited the effective arch area to the top and bottom flanges, involving a serious waste of material.

To remedy this defect a new arch was patented in about 1890, known as the "Lee" arch, in which the voids ran

parallel to the line of thrust, or at right angles to the supporting beams. One of these arches is shown in Fig. 42, and it will be seen that the effective area now comprises the vertical webs, as well as the horizontal ribs; in other words, all of the material performs useful work *as an arch*. A further improvement was attempted by the use of a porous terra-cotta, made from a fire-clay which, before it is burned, is mixed with saw-

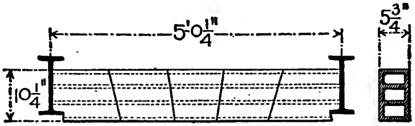

Fig. 42.—The "Lee" Terra-cotta Flat Arch.

dust and finely cut straw. These ingredients are consumed during the firing, leaving the material in a very porous condition, and thus greatly reducing the dead weight of the arch itself. A comparison of the weights of the old Pioneer and the newer Lee arch may be made as follows (weight given is per square foot):

	Pioneer.	Lee.
9″ arch................	33 lbs.	25 lbs.
10″ "	37 "	30 "
12″ "	40 "	35 "
15″ "		40 "

Another step of progress lay in the skew-back or butment pieces, which gave a better bearing against the beam webs by means of intermediate cross-ribs, as well as by the top and bottom flanges.

Denver Tests.—Some very interesting and valuable tests of fireproof floor arches built after the Pioneer and Lee methods were published in No. 796 of the *American Architect and Building News*—undoubtedly forming one of the most satisfactory and extensive series of public tests yet attempted on such construction. The trials were made in Denver, Col.,

1890, for the Denver Equitable Building Company, under the supervision of a board of architects. The arches were sprung from beams placed 5 ft. centres, as shown in Fig. 42, and the conditions included static loading, a drop test, a fire and water test, and a continuous fire test.

In the test for static loads the Lee arch deflected gradually under the increased weights to .065 of a foot, sustaining a final load of 15,145 lbs. for two hours. The Pioneer arch gave way suddenly at the haunches under a load of 5,429 lbs.

In the drop test a piece of wood 12″ × 12″ × 4′ was let fall from a height of 6 ft. The Pioneer arch was shattered at the first blow, while the Lee arch, under the same test, stood up to the eleventh drop, the former blows shattering but parts of the arch.

In the fire and water tests, three applications of water combined with fire destroyed the Pioneer arch, while the Lee arch received eleven applications of water, and at the end of twenty-three hours remained practically uninjured, requiring eleven blows from the ram to break it.

In the continuous fire test the fire was maintained continuously beneath a Lee arch for twenty-four hours, and the arch then supported a load of bricks of 12,500 lbs. on a space 3 ft. wide in the central portion of the arch.

Considering the static loads, the results may be better judged as follows:

	Pioneer.	Lee.
	lbs.	lbs.
Breaking-load per square foot of 9 sq. ft. loaded area.	603	1670
Reduced to equally distributed load, 3′ 0″ × 5′ 0″..........	360	1008
Assumed load per square foot, as occurring in practice.....	150	150
Coefficient of safety.....	2.4	6.7

Manufacture of Terra-cotta Arch-blocks.—Terra-cotta floor arches now in common use are made of either "porous," "semi-porous," or "hard-burned" terra-cotta. These desig-

nations are indicative of the methods employed in the manufacture of the clay.

Porous terra-cotta, sometimes called cellular pottery, soft tile, porous tile, or terra-cotta lumber, may be briefly described as consisting of pure clay mixed with sawdust or finely cut straw. This mixture is passed through the "tile-machines," where the blocks are manufactured to the required form, after which they are placed in dry rooms for a sufficient time to permit of handling, the final burning or hardening being accomplished in kilns where a temperature of from 2,100 to 2,500 degrees is maintained for from three to four days. The sawdust or straw in the clay is completely consumed during the firing, thus leaving the finished product in a honey-combed or porous state, thereby reducing the weight of the original mass.

Porous terra-cotta can be readily cut with ordinary tools, and the blocks are often soft enough to receive nails or screws used in applying the interior trim. Such nailing blocks are usually made solid.

Semiporous terra-cotta differs from that of the porous variety principally in the composition of the mixture. The ingredients are usually fire-clay containing about 60 per cent. of silica, coarsely ground calcined fire-clay, and coarsely ground bituminous coal. The resulting product is slightly more porous than the best grades of fire-brick, but not as soft as porous terra-cotta.

Hard-burned terra-cotta, also termed hard tile, or dense tile, is made of pure clays, without the addition of any combustible materials. During its manufacture, the clay is subjected to a high pressure, thus giving the material a dense texture, and great strength under crushing loads. Hard-burned terra-cotta cannot be readily cut, but must be broken, and as the material is brittle, it is unreliable under shocks or suddenly applied loads.

Construction of Flat Terra-cotta Arches.—Flat arches, constructed of terra-cotta blocks, are composed of two "skew-backs," "skews," or "butment pieces," which bear against the beam webs and fit around the lower flanges of the beams; one centre block or "key," and "fillers," "part-fillers," or "intermediates" which fill the spaces between the skew-backs and key. In end-construction, a filler or whole intermediate block is usually considered as 12 ins. long, a part-filler being less than this in length. In side-construction the lengths of the fillers vary according to the manufacturers' practice.

All types of flat arches are usually made with bevelled joints—that is, all of the joints in each half of the arch are made parallel to the side of the key. Radial joints, or such as would meet at a common centre if prolonged, are occasionally employed, and these make an arch better and stronger, and more theoretically correct, but the increased number of shapes required for arches of varying span, makes the cost of manufacture almost prohibitory.

The protection of the bottom flanges of the beams is usually made by introducing separate strips of terra-cotta, or "beam-facings," which are held in place under the beam flanges by means of bevelled lips on the skew-backs, as shown in Fig. 41. Some manufacturers have dispensed with separate beam-facings, substituting therefor projecting lips made on, and as a part of, the skew-backs, the lips from the two skew-backs meeting at the centre line of the beam as shown in Fig. 49. But in manufacturing such skews, with the beam protections made as a part of the blocks, these flanges were so liable to deformation by warping in the drying or burning that the skews could often not be placed upon the beams without breaking the flange from the block. The majority of manufacturers have consequently abandoned this method, and separate "beam-facings" are now generally used.

The arches are set on "centres" of plank (hung from the

beams by hook-bolts), which should remain in place at least forty-eight hours in good dry weather, and considerably longer in damp or wet weather. Clear cement mortar should preferably be used, many of the blocks being "scored" or grooved on the outer surfaces as shown in Fig. 43, to provide a better key for the mortar in the joints, and for the plastered ceiling.

The depth of the terra-cotta arch-blocks depends upon the span and the load to be carried. The maximum spans for the varying depths of blocks under specified loads per square foot are usually furnished by the manufacturer, and for ordinary requirements such data will generally be found reliable if furnished by responsible firms. A safe rule for ascertaining the allowable span for any depth of arch-block is that the maximum span in feet should not exceed two-thirds the depth in inches of the arch-block employed.

Present Types of Terra-cotta Arches.—Flat terra-cotta arches now in ordinary use include "side-construction" arches, "end-construction" arches, and "combination" arches made of part side and part end methods.

Side-construction Arches are made of blocks in which the voids run parallel to the supporting beams, as in the early forms of Pioneer arches, before illustrated. A side-construction arch with bevelled joints is shown in Fig. 43. This

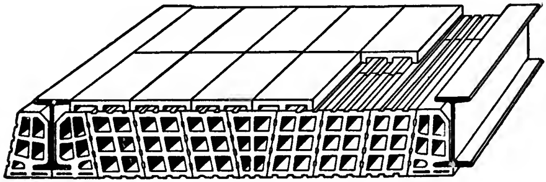

FIG. 43.—Side-construction Terra-cotta Arch. Bevelled Joints.

represents a deep arch, the blocks of which have one vertical and two horizontal interior webs or partitions. Shallower

arches have less interior webs, one horizontal web or partition being generally used for 6-in., 7-in., or 8-in. blocks, two webs for 9-in., 10-in., and 12-in. blocks, and three or four webs for 15-in. and 18-in. blocks.

The average permissible spans and weights per square foot for arches of this type are as follows:

Depth of Arch.	Width of Span.		Weights per sq. ft. in lbs. Hard-burned.	Porous.
6 ins.	3 ft.	to 4 ft.	27	25
7 ins.	4 ft.	to 4 ft. 6 ins.	29	26
8 ins.	4 ft. 6 ins.	to 5 ft.	32	28
9 ins.	5 ft.	to 6 ft.	37	32
10 ins.	6 ft.	to 6 ft. 6 ins.	40	36
12 ins.	6 ft. 6 ins.	to 7 ft.	44	40

Side-construction arches are made of both hard-burned and porous terra cotta.

A side-construction arch with radial joints and segmental interior webs is shown in Fig. 44. This arch is made in 8-,

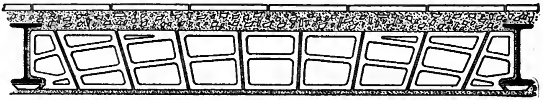

FIG. 44.—Side-construction Terra-cotta Arch. Radial Joints.

9-, 10-, and 12-in. depths, weighing respectively 28, 29, 35, and 46 lbs. per square foot.

End-construction Arches are made of blocks in which the voids run at right angles to the beams, or from beam web to beam web. The skew-back pieces are of the same general form as the intermediate blocks, but are made to fit against the beam web and flange, without, however, any continuous bearing-surface, as is obtained in the side-construction skew. The vertical and horizontal webs and partitions run directly to the beam, and as these are the load-bearing areas, a stronger

and better skew is obtained. The skew-backs are made with dovetailed lips to hold the beam-facings in place. These arches are usually made of porous terra-cotta and always with bevelled joints.

Fig. 45 shows an end-construction arch of porous material

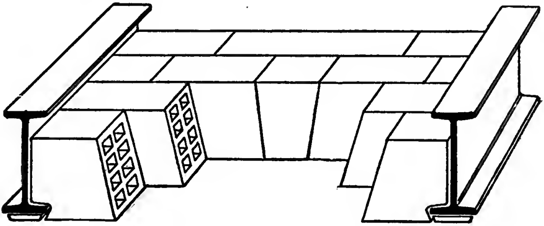

FIG. 45.—End-construction Terra-cotta Arch.

in which the blocks break joints with those in adjacent arches, each arch being continuous from beam to beam. This is considered the best practice. The depth of arch-blocks varies from 6 ins. to 15 ins. Maximum spans and average weights per square foot, set in position, are as follows:

Depth of Arch.	Maximum Span.	Weight per sq. ft.
6 ins.	4 ft. 6 ins.	29 lbs.
8 ins.	5 ft. 6 ins.	31 lbs.
9 ins.	6 ft.	32 lbs.
10 ins.	6 ft. 6 ins.	33 lbs.
12 ins.	7 ft.	39 lbs.
15 ins.	8 ft.	46 lbs.

An end-construction arch intended for extremely heavy service has been introduced by the Pioneer Fireproof Construction Co., of Chicago, the arch-blocks being made of 15-in., 16-in., 18-in., and 20-in. depths of the form shown in Fig. 46. This type of arch with recesses or voids between the individual blocks affords a very stiff floor, due to the increased

depth, and yet at no increase in weight, while a further advantage is gained in permitting the tie-rods to span the bays with-

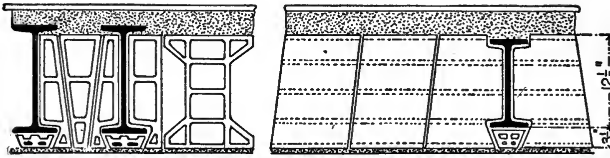

Fig. 46.—End-construction Terra-cotta Arch, "Pioneer" Type.

out cutting into the blocks. The span lengths and weights as given by the manufacturers are as follows:

Depth of Arch.	Maximum Span.	Weight per sq. ft.
15 ins.	8 ft. 0 ins.	38 lbs.
16 ins.	——	42 lbs.
18 ins.	——	50 lbs.
20 ins.	12 ft. 0 ins.	56 lbs.

This arch is a development of the form shown in Fig. 47.

Combination End- and Side-construction Arches are formed of side-construction skew-backs, and end-construction intermediates, the combined use being largely due to the greater ease with which side-construction skew-backs can be set, while the intermediate blocks may still be retained of the superior end-construction type.

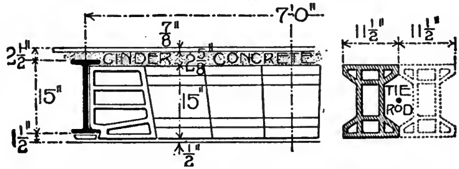

Fig. 47.—Johnson Type of Terra-cotta Arch.

One of the first combination arches was as shown in Fig. 47. This was known as "Johnson's patent flat arch," and

this type has been used extensively in many of Chicago's largest buildings.

Fig. 48 illustrates a combination arch made in 8-, 10-,

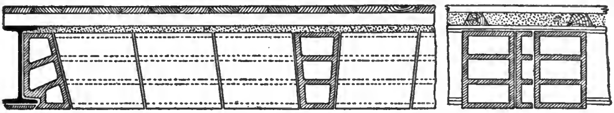

FIG. 48.—Combination Terra-cotta Arch.

11-, and 12-in. depths, weighing respectively 27, 34, 36, and 41 lbs. per sq. ft.

The "Excelsior" combination arch is shown in Fig. 49.

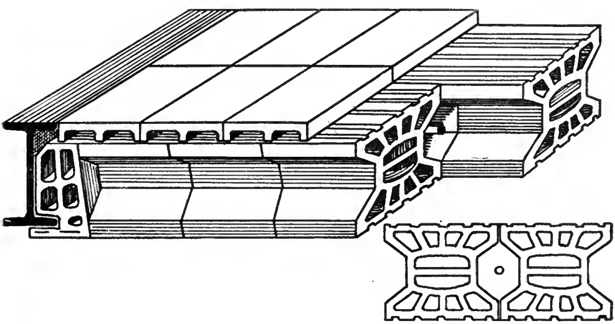

FIG. 49.—Combination Terra-cotta Arch, "Excelsior" Type.

This also, like Figs. 46 and 47, possesses the recessed sides or voids between the arch-blocks, which, while reducing the weight, permit a free passage for the tie-rods. The following spans and weights per square foot are given by the manufacturer:

Depth of Arch.	Safe Span.	Weight per sq. ft.
8 ins.	5 ft. to 6 ft.	27 lbs.
9 ins.	6 ft. to 7 ft.	29 lbs.
10 ins.	7 ft. to 8 ft.	33 lbs.
12 ins.	8 ft. to 9 ft.	38 lbs

Segmental Terra-cotta Arches are usually limited to use in warehouses, factories, or breweries, where heavy floor-loads have to be carried regardless of the ceiling appearance. In office or mercantile buildings, a flat ceiling is desirable on account of appearance and the greater light reflected from an unbroken plane.

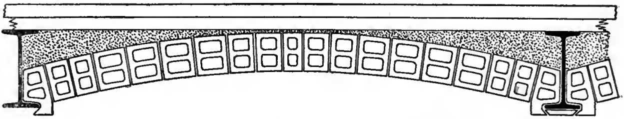

FIG. 50.—Segmental Terra-cotta Arch.

Segmental arches are usually made of side-construction blocks, 4, 5, 6, or 8 ins. square, and about 12 ins. long. Both porous and hard-burned materials are used. The spans employed vary from 5 ft. to 20 ft., and the rise should never be less than one inch per foot of span, or preferably one and one-half inches per foot. The usual form is shown in Fig. 50, for which the spans and weights per square foot, exclusive of concrete filling and plastering, will average about as follows:

4-in. blocks,	8-ft. span,	16 lbs. per sq. ft.
6-in. blocks,	16-ft. span,	26 " " "
8-in. blocks,	20-ft. span,	28 " " "

The skew-back blocks should be either very heavy or entirely solid, and the concrete levelling should be of good quality and levelled up to a point at least one inch above the crown. The concrete at the haunches is sometimes made with voids, as in Fig. 51.

Raised skew-backs with flat arches are often employed, as in Fig. 52. These are frequently used in roof construction, where long and deep beams are necessary, but where the arch depth may be reduced on account of lighter floor-loads per square foot.

Filler blocks of terra-cotta are sometimes used instead of the usual concrete filling over the arches. These are to decrease the weight. See Fig. 49.

Choice of Terra-cotta Arch.—As to a choice between the various forms and materials in which terra-cotta arches are manufactured, the reader is referred to the author's "Fireproofing of Steel Buildings," in which volume a complete

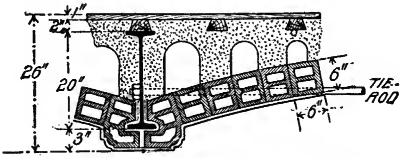

FIG. 51.—Segmental Terra-cotta Arch.

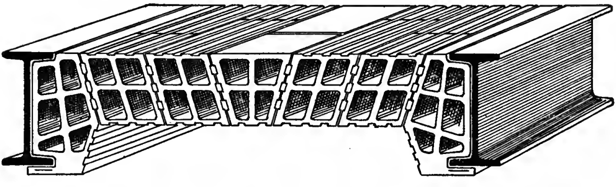

FIG. 52.—Side-construction Terra-cotta Arch. Raised Skew-backs.

discussion will be found relative to all ordinary forms of terra-cotta, concrete, and composition floors.

Briefly, it may be stated that a porous terra-cotta end-construction arch, with thick webs, well rounded interior corners, level soffit, and of the full depth of the beams, will best answer all requirements as to load-bearing capacity, shock, and fire- and water-tests.

Concrete and Composition Floors.*—The widespread interest displayed in the subject of fireproof floors is well indi-

* For complete descriptions as to the construction, setting, comparative advantages and disadvantages, and fire-resisting qualities, etc., see the author's "Fireproofing of Steel Buildings," John Wiley & Sons, N. Y. 1899.

cated by the numerous types which have entered the field in competition with the hollow-tile flooring. These newer systems differ greatly in principle, and while many of them are founded on sound constructive practice, others are open to serious question and should be used with much discrimination. While it is no difficult matter to construct a floor of concrete or various compositions which will be of sufficient strength and possess apparently satisfactory fire-resisting qualities, it is still not so easy to secure a minimum cost, a minimum weight, and a minimum of repair made necessary by possible fire and water exposure.

Only the more ordinary and commendable forms of concrete and composition floor systems will here be described.

The most widely known forms of concrete floors include the Roebling, Columbian, and Expanded Metal Company's floors.

Roebling Floors.—The concrete floors made by the John A. Roebling's Sons Company include three distinct forms, viz., a concrete arch with exposed soffit, a flat construction somewhat similar to the Columbian floor (made of metal bars and a concrete plate, with a suspended ceiling beneath), and a concrete arch with suspended ceiling. The latter is the most common form, and is illustrated in Fig. 53.

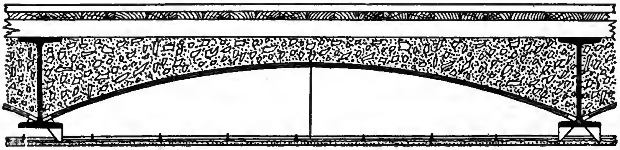

FIG. 53.—Roebling Concrete Floor Arch with Suspended Ceiling.

The arch is formed on a permanent arched centering made of wire cloth stiffened with $\frac{3}{8}$-in. to $\frac{1}{2}$-in. diameter steel rods woven into the cloth about 9-ins. centres. These wire centres

are made of the proper size and form at the factory, and in erection the sheets are lapped at the joints, and securely laced.

A cinder-concrete arch (generally made of 1 part Portland cement, 2½ parts sand, and 6 parts clean anthracite coal cinders) is then filled in up to the tops of the beams, giving a thickness of not less than 3 ins. at the crown of the arch.

A suspended ceiling is made by attaching flat bars, spaced about 16-in. centres, to the under sides of the I-beams by means of patent clamps. Stiffened wire lathing is then laid at right angles to, and on the under sides of these bars, the laps being laced with galvanized wire. In spans over 3 ft. 6 ins., the ceiling is further supported by means of wire hangers dropped from the crown of the arch about 30-in. centres, which fasten to a $\frac{5}{16}$-in. diameter steel rod laid over and laced to the ceiling bars.

Permissible spans, with their attendant weights, will average about as follows:

Depth of Beams or Thickness of Concrete at Haunches.	Maximum Span.	Thickness of Crown at Centre of Arch.	Weight per sq. ft. Including Concrete and Wire Centering.
8 ins.	4 ft. 0 ins.	3 ins.	33 lbs.
9 ins.	4 ft. 6 ins.	3 ins.	34 lbs.
10 ins.	5 ft. 0 ins.	3 ins.	36 lbs.
12 ins.	6 ft. 0 ins.	3 ins.	41 lbs.
15 ins.	7 ft. 6 ins.	3 ins.	47 lbs.

Many tests have shown remarkable strength qualities for this arch form, and fire- and water-tests have demonstrated generally satisfactory fireproofing qualities; but any system of fireproofing which relies entirely upon a suspended ceiling for the insulation of the beam flanges is not, in the author's opinion, to be very highly recommended. Such ceilings will undoubtedly protect the beams to a large extent, but the ceilings will fail under severe conditions, and possibly too early to

save the beams from collapse, while even the reconstruction of the ceilings would form a large item in repairs.

A still more satisfactory form of the Roebling floor is the concrete arch with exposed soffit, where the form is the same as that previously shown in Fig. 53, except that the curved soffit is left exposed, and the lower flanges of the beams are surrounded by wire lathing and concrete of semicircular form. But as level ceilings are considered a requisite in office or dwelling buildings, this type has generally been limited to factories, warehouses, breweries, etc.

Columbian Floor.—The concrete floor manufactured by the Columbian Fireproofing Company is of the flat or plate construction, consisting of a combination of rolled-steel bars and concrete. See Fig. 54. The bars are suspended from

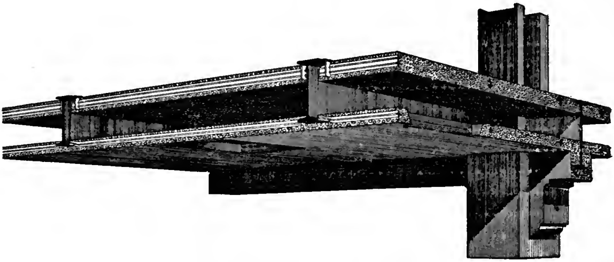

FIG. 54.—Columbian "Flat-ceiling" Floor Construction.

the upper flanges of the floor-beams by means of steel stirrups, which are perforated to the shape of the bars employed. A concrete plate is then filled over a temporary centering, the top of the concrete coming flush with the upper flanges of the beams. The mixture usually employed for all but the very heaviest construction is composed of 1 part Portland cement, 2½ parts sand, and 5 parts broken stone. Bars 2 ins. deep are generally used for hotels or office buildings, 1½-in. bars for residences and apartment houses, and 2½-in. sections for ware-

houses, storage, and mercantile buildings. The bars are spaced from 20- to 24-ins. centres.

The ceiling slab is made of 1-in. bars of the same form, which rest on the lower flanges of the beams. These support a cinder-concrete ceiling slab, made of 1 part Portland cement, 5 parts cinders, and $2\frac{1}{2}$ parts sand.

The beam-webs are either left exposed, or are encased in concrete. The flat-ceiling construction is not usually employed for spans exceeding 7 ft., while in the case of less span, and light loads, the bars are sometimes made to pass directly over the floor-beams, resting on them, thus dispensing with the stirrups.

A panelled construction is also made by the same company, this being like the former type as far as the floor-plate is concerned, but without the ceiling-plate. In this type, the beams are encased in concrete, thus showing a panelled construction from below. This is more applicable to warehouse or mercantile building construction.

Size of Bars. Inches.	Thickness of Floor. Inches.	Panelled Construction. Solid Casing.		Flat Ceiling Construction.	
		Stone-Concrete. Pounds.	Cinder-Concrete. Pounds.	Stone-Concrete. Pounds	Cinder-Concrete. Pounds.
1	$2\frac{1}{4}$	42	$26\frac{1}{2}$	$48\frac{1}{2}$	37
$1\frac{1}{2}$	$2\frac{3}{4}$	42	$26\frac{1}{2}$	$48\frac{1}{2}$	37
2	$3\frac{1}{4}$	46	29	$54\frac{1}{2}$	$40\frac{1}{2}$
$2\frac{1}{2}$	$3\frac{3}{4}$	54	$35\frac{1}{2}$	$59\frac{1}{2}$	$43\frac{1}{2}$

The Columbian floors are very satisfactory as to strength, and acceptable as to fire-resisting qualities, although the stone-concrete employed is inferior to cinder-concrete under fire-tests. As regards corrosive influences, however, stone-concrete is to be preferred to cinder mixtures.

Expanded Metal Co.'s Floors.—Several types of concrete floors are manufactured by the various companies acting as licensees from the Expanded Metal Company.

The floor shown in Fig. 55 is employed for spans under 8 feet, either with or without a suspended ceiling. A wooden centering is employed, suspended from the beams at a proper level to receive the concrete plate. Expanded metal is then stretched lengthwise across the beams in sheets, and concrete is spread to form a slab about 3 ins. thick for ordinary floors, this being tamped so that the expanded metal becomes embedded in the lower inch of the floor plate. The concrete is usually made of 1 part cement, 2 parts sand, and 6 parts furnace-cinders, weighing about 84 lbs. per cu. ft.. Cinder filling

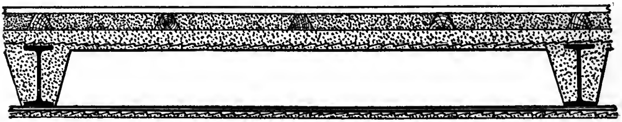

FIG. 55.—Expanded Metal Co.'s Floor with Suspended Ceiling.

is placed between the floor screeds, weighing about 60 lbs. per cu. ft.. If a suspended ceiling is desired, small channels or angles spaced 12- to 16-ins. centres are attached to the bottoms of the beams by means of malleable-iron clips. Expanded metal is then fastened to these, ready for the plastering.

A still different form, which corresponds very closely to the Roebling floor, is shown in Fig. 56. In this case sheets of expanded metal are sprung between the beam flanges as a

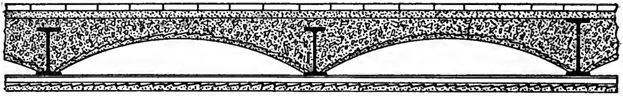

FIG. 56.—Expanded Metal Co.'s Concrete Arch.

permanent centering to receive the concrete arch. This type is adapted to heavier loads than the floor shown in Fig. 55, but it should only be employed when the span is narrow enough to permit of a central height of $\frac{1}{6}$ to $\frac{1}{8}$ of the span.

Fireproof floors employing expanded metal have been very extensively used, and have generally met with much favor. They are easily adapted to all conditions of framing, and are comparatively light and reasonable in cost; but, except in the arched form shown in Fig. 56, the reliance for tensile strength, in employing concrete as a beam, is placed upon the thin sheets of expanded metal, the ultimate life of which, surrounded by cinder-concrete with corrosive tendencies, is open to serious question. If the expanded metal were always thoroughly encased in *cement mortar* at the bottom of the concrete plate, these forms would be far more commendable.

Metropolitan Floor.—The Metropolitan system of fireproof floors is illustrated in Fig. 57. Wire suspension-cables

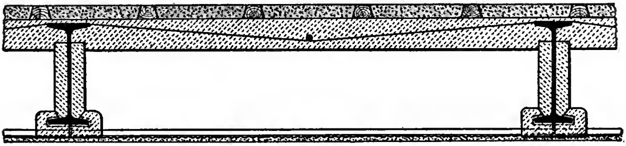

FIG. 57.—Metropolitan Floor. Flat Ceiling Construction.

are used for the supporting members, each consisting of two No. 12 galvanized wires, twisted together and laid across the tops of the beams with hooks or anchors where they terminate at the end beams or walls. These cables are spaced from $\frac{7}{8}$-in. to $1\frac{1}{2}$-in. centres, according to spans and loads. They are laid parallel, and are then depressed slightly in the centre of each bay by means of $\frac{7}{8}$-in. round iron rods which are laid lengthwise on the cables so as to cause a uniform sag of about 2 ins. below the tops of the I-beams. Wooden centres are placed between the beams and about 1 in. below the iron rods, and a composition formed of 1 part plaster of Paris to 2 parts of shavings, with sufficient water to form a plastic mass, is then poured in place and tamped to a level of about $\frac{1}{2}$ in. above the

tops of the beams. The floor-plate is thus about 4½ ins. thick, not including the screeds or finished flooring.

The suspended ceiling is made by clipping ¾-in. by ¼-in. flat bars to the lower flanges of the I-beams, ready to receive wire lathing. The beam webs and flanges are also protected by the same mixture, as shown in the illustration.

This system has also been extensively used in fireproof structures, and its decided lightness forms a great advantage in certain instances. The load-carrying qualities are also excellent, but disadvantages occur from discoloration due to the sap in the shavings employed, and from the sometimes uneven drying of the mass. The fireproof qualities are fairly acceptable. The suspended ceilings when exposed to fire and water are apt to be thoroughly destroyed, and the floor-plate will be partly washed away or rendered soft at the surface, but reconstruction is comparatively simple after such possible injury.

Selection of Floor Type.—The above-mentioned concrete and composition floors may not include *all* of the commendable types now on the market, but as before stated, those described constitute the better-known examples of trustworthy character. Several other so-called fireproof floors have been used to a considerable extent, many of them in very important structures, but their ultimate value will not always bear close investigation. It will be noticed that little has been said as regards the comparative cost of the types of terra-cotta and concrete floors here mentioned. This question will undoubtedly serve as a prime factor in making a choice between the various methods, but the question of *first cost* can in nowise be taken as a guarantee of ultimate wisdom. The true value of a construction can only be determined by the tests of time and destructive elements, such as corrosive action, and fire- and water-tests. These considerations, with practical questions as to weight, depth, form, and a minimum of repair required by

possible damage under fire and water, should govern a selection, regardless, in so far as may be judicious, of the first cost. Almost all of the better-known floor-systems, whether terra-cotta, concrete, or composition, will, under all ordinary conditions, show a reasonable factor of safety under usual loads. As to choice of concrete *vs.* terra-cotta floors, the best constructors agree that either type may be made perfectly satisfactory if well designed and executed, while both are equally bad where defective design, materials, or workmanship are employed.

Building Laws—Floor Arches.—The following requirements are specified in the Chicago building ordinance, Section 108: "The filling between the individual iron or steel beams supporting the floors of fireproof buildings shall be made of brick arches, or concrete arches, or hollow-tile arches. Brick arches shall not be less than 4 ins. thick, and shall have a rise of at least $\frac{3}{4}$ in. to each foot of span between the beams. If the span of such arches is more than 6 feet, the thickness of the same shall not be less than 8 ins. If hollow-tile arches having a straight soffit are used, the thickness of such arches shall not be less than at the rate of 2 ins. per each foot of span. If concrete arches are used, the concrete in the same shall not be strained more than 100 lbs. per sq. in., if the concrete is made of crushed stone, nor more than 50 lbs. per sq. in., if the concrete is made of cinders. In all cases, no matter what the material or form of the arches used, the protection of the bottom flanges of the beams and so much of the web of the same as is not covered by the arches shall be made as before specified for the covering of beams and girders."

Also, Section 90: "Hollow tile and porous terra-cotta may be used in the form of flat arches for the support of floors and roofs; such floor arches having a height of at least 2 ins. for each foot of span. The arches must be so constructed that the joints of the same point to a common centre; the butts of the arches shall be carefully fitted to the beams supporting

them; and there shall be a cross-rib for every 6 ins. or fractional part thereof in height; and in addition to these there shall also be diagonal ribs in the butts. Floor arches made in the form of a segment of a circle or ellipse must be constructed upon the same principle. Such arches, whether flat or curved, shall have their beds well filled with mortar, and the centres shall not be struck until the mortar has been set."

The Building Code of Greater New York specifies that fireproof floors shall consist of: segmental brick arches, flat terra-cotta arches (of a depth not less than $1\frac{3}{4}$ ins. per foot of span, not including any projection of the arch below the under side of beams); segmental terra-cotta arches (with the depth of arch-blocks not less than 6 ins., and with a rise of not less than $1\frac{1}{4}$ ins. per foot of span); segmental Portland cement concrete arches (the thickness at crown of arch to be not less than 6 ins., the rise to be not less than $1\frac{1}{4}$ ins. per foot of span, with the soffit reinforced with some form of metal weighing not less than one pound per square foot, and with no openings larger than 3 ins. square); or "various fillings" subject to fire- and water-tests as per the following requirements:

"Or between the said beams may be placed solid or hollow burnt clay, stone, brick, or concrete slabs in flat or curved shapes, concrete or other fireproof composition, and any of said materials may be used in combination with wire cloth, expanded metal, wire strands, or wrought-iron or steel bars; but in any such construction and as a precedent condition to the same being used, tests shall be made as herein provided by the manufacturer thereof under the direction and to the satisfaction of the Board of Buildings, and evidence of the same shall be kept on file in the Department of Buildings, showing the nature of the test and the result of the test. Such tests shall be made by constructing within inclosure walls a platform consisting of four rolled-steel beams, 10 ins. deep, weighing each 25 lbs. per lineal foot, and placed 4 ft. between the

centres, and connected by transverse tie-rods, and with a clear span of 14 ft. for the two interior beams and with the two outer beams supported on the side walls throughout their length, and with both a filling between the said beams and a fire-proof protection of the exposed parts of the beams of the system to be tested, constructed as in actual practice, with the quality of material ordinarily used in that system, and the ceiling plastered below as in a finished job; such filling between the two interior beams being loaded with a distributed load of 150 lbs. per sq. ft. of its area and all carried by such filling; and subjecting the platform so constructed to the continuous heat of a wood fire below, averaging not less than 1,700 degrees Fahrenheit for not less than four hours, during which time the platform shall have remained in such condition that no flame will have passed through the platform or any part of the same, and that no part of the load shall have fallen through, and that the beams shall have been protected from the heat to the extent that after applying to the under side of the platform at the end of the heat-test a stream of water directed against the bottom of the platform and discharged through a $1\frac{1}{8}$-in. nozzle under 60 lbs. pressure for five minutes, and after flooding the top of the platform with water under low pressure, and then again applying the stream of water through the nozzle under the 60-lbs. pressure to the bottom of the platform for five minutes, and after a total load of 600 lbs. per sq. ft. uniformly distributed over the middle bay shall have been applied and removed, after the platform shall have cooled, the maximum deflection of the interior beams shall not exceed $2\frac{1}{2}$ ins. The Board of Buildings may from time to time prescribe additional or different tests than the foregoing for systems of filling between iron or steel floor-beams, and the protection of the exposed parts of the beams. Any system failing to meet the requirements of the test of heat, water, and weight, as herein prescribed shall be prohibited from use in any building.

hereafter erected. Duly authenticated records of the tests heretofore made of any system of fireproof floor filling and protection of the exposed parts of the beams may be presented to the Board of Buildings, and if the same be satisfactory to said Board, it shall be accepted as conclusive."

The above section of the New York Building Code gives substantially the test conditions required in the fire- and water-tests on various fireproof floors, made by the New York Building Department in 1896. A detailed description of the test-kilns, method of testing, and results as to the Rapp, Roebling, Thompson, M'Cabe, Columbian, Bailey, Clinton, Wire-cloth, Manhattan, Expanded Metal Company's, Metropolitan, Fawcett, Guastavino, and Terra-cotta floors, is given in the author's "Fireproofing of Steel Buildings."

Floor Loads.—Before considering the details of floor-beams and girders, the question of loads, which will largely govern the design of the floor-system, must be examined.

Loads occurring in building construction may be classified as live, dead, wind, and eccentric loads. These will all be considered in their proper places. The principal loads affecting the floor-system are:

Live Loads, comprising the people in the building, office furniture, movable stocks of goods, small safes, elevator and tank loads, or varying loads of any character. Large safes require especial provision for support.

Dead Loads, comprising all of the static loads due to the constructive parts of the building (such as floors, roofs, walls, columns, etc.), stationary machinery, and any other permanent loads.

Live Loads.—The live loads to be provided for in the design of the floor-system are usually specified in the local building ordinances, according to the purposes for which the building is intended. The designer is therefore limited by the requirements under which he is obliged to work.

For office buildings, both the Chicago and Boston laws require a unit of 100 lbs. per sq. ft., while the New York law specifies live loads of 75 lbs. per sq. ft. for the upper floors, and 150 lbs. for the first floor. In the author's opinion, all of these requirements are excessive, providing proper restrictions are enforced as to wind-strains, heavy safes, and vibratory influences due to printing or manufacturing.

Without reference to building laws, the following live loads have always been considered standard practice:

For floors of dwellings..........	40 lbs. per sq. ft.
For dense crowd of people.......	80 " " "
For theatres, churches, etc.......	80 " " "
For ball-rooms or drill-halls......	90 " " "
For warehouses, etc............	from 250 lbs. up.
For factories..................	200 to 450 lbs.

While 80 lbs. is the maximum possible live load per square foot from a crowd of people (unless dancing be considered), still we can hardly expect to realize any such load under the conditions governing an office building. Large crowds very seldom collect in offices, except, perhaps, on the two or three lower floors devoted to stores or banking purposes, and greater allowances are generally made for such places. The ordinary office furniture will certainly not exceed, and seldom equal, the weight allowed for persons, and hence additional security is introduced.

A very valuable and interesting article in the *American Architect*, August 26, 1893, gives the results of some experiments made by Messrs. Blackall & Everett, Boston architects, on the actual weights of all moving loads in some of the larger Boston office buildings. The loads considered were those due to people and all possible movable articles, including all office fittings except such as were a part of the floors or partitions, radiators excepted. The results were as follows: In 210

offices in the Rogers, Ames, and Adams buildings, an average of 16.3 lbs. per sq. ft. was found for the Rogers Building, 17 lbs. for the Ames, and 16.2 lbs. for the Adams Building. The *greatest* moving load in any one office in the three buildings was 40.2 lbs. per sq. ft., while the *average* for the heaviest ten offices in each of these buildings was 33.3 lbs. per sq. ft. Mr. Blackall concludes: "If these figures are to be trusted to any extent whatever, then even under the most extreme circumstances, taking the pick of the heaviest offices in the city and combining them into one tier of ten stories, the average load per square foot would be only a trifle over 33 lbs., while for all purposes for strength an assumption of 20 lbs. would be amply sufficient in determining the loads on the foundations, as well as on the columns of the lower stories."

These experiments plainly indicate that a live load of 40 lbs. per sq. ft. is amply sufficient for office areas, and, where not restricted by building laws, 35 and 40 lbs. per sq. ft. have been used as the assumed live loads in many important and very satisfactory modern office buildings.

In the Mills Building, erected in San Francisco in 1891, the live loads were as follows:

	Beams.	Girders.	Columns.	Footings.
First floor.	60	50	40	—
Second floor to attic . . .	40	30	20	—
Roof.	20	15	10	—
Rotunda.	60	50	40	—

In the Venetian Building in Chicago the beams were calculated for the following live loads:

Upper floors.	35 lbs.
Second, third, and fourth floors.	60 "
First floor.	80 "

Girders carry 80 per cent., columns 50 per cent.

Mr. E. C. Shankland, who has designed and superintended the construction of a large number of the most prominent high buildings in Chicago and elsewhere, states that "the live load, consisting of the weight of the tenants, the furniture, and the partitions, which are frequently changed, is taken between 60 lbs. and 75 lbs. per sq. ft. for the upper floors of an office building, and between 75 lbs. and 100 lbs. per sq. ft. for the first and second floors, which are generally used for shops and banks. The weight of the tenants and furniture of a typical office have been found by experiment to be only 6 lbs. or 7 lbs. per sq. ft.; it certainly does not exceed 12 lbs. The average weight of the partitions is 25 lbs. per sq. ft. of floor." *

Deducting, then, the partition load of 25 lbs. per sq. ft. from the live loads recommended by Mr. Shankland, the live loads as given above become 35 lbs. to 50 lbs. per sq. ft.

The small live loads found, by actual experiment, to exist in office areas, such as 12 lbs. per sq. ft. according to the last data quoted, and 16 or 17 lbs. per sq. ft. according to Mr. Blackall's article, have tempted the use of unit loads as low as 20 lbs. per sq. ft.; but such recommendations are to be seriously questioned, and even heartily condemned in conservative practice.

While 20 lbs. per sq. ft. may be amply sufficient for average loads at present, we must remember that the use of an average is always dangerous, while provision should be made, but not recklessly, for all possibilities of extremes, either present or future. For it must be remembered that the character of a building's contents is very liable to extreme change. The entire building, or possibly only portions thereof, may be devoted to very different uses from those primarily assumed, so that it becomes a very nice problem to

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* See Minutes of Proceedings of The Institution of Civil Engineers, vol. cxxviii.

balance present economy with maximum present requirements or future possibilities. The present live load per square foot may not always be taken as the maximum occurring during the life of the building. Most building ordinances provide against radical change in the character or degree of the floor-loads, and against the introduction of vibratory or manufacturing elements not provided for in the original design. But the line is sometimes difficult to draw, and, as in the strength of materials, a sufficient factor of safety should always be employed.

If the building is to be used for the purposes of printing or manufacturing, the assumed live loads must be substantially increased to care for the vibration always induced by the pulsations of machinery, the pull on belts, and especially the shocks due to the starting and stopping of all dynamic forces.

For purely office purposes, however, it would seem that the present requirements of the Chicago and Boston building laws, and even of the New York law, are too high. Live loads of 80 lbs. per sq. ft. for the lower or busier floors, and 40 lbs. per sq. ft. for the upper or office floors, are certainly safe and ample, and good averages, considered in all lights. But while the live loads per square foot might be reduced to these figures over large areas in proportioning the metal-work, the maximum possible live load should still be used when any single floor arch is considered by itself, or subjected to tests to determine its strength. For the working factor of safety required of terra cotta or concrete-floor constructions, (which may be considered as forming the poorest class of masonry construction), should be considerably greater than the factor of safety required in as reliable a material as steel. Rankine advises the use of $\frac{1}{3}$ to $\frac{1}{6}$ the ultimate strength in metals, $\frac{1}{6}$ to $\frac{1}{10}$ in wood, and $\frac{1}{4}$ to $\frac{1}{8}$ in masonry.

The live loads recommended, viz., 40 and 80 lbs. per sq. ft., are independent of the partition loads. Partitions are

sometimes classed as live loads, because liable to change in location, as is made possible by present constructions, while in other cases they are assumed as a portion of the dead load. The present New York and Chicago laws both require partitions to be considered as a part of the dead load.

Prior to the enactment of the last Building Ordinance of the City of Chicago, March, 1898, practice was well defined in the matter of decrease of live loads per square foot, as they are transferred from beams to girders, from girders to columns, and thence down the columns to the footings. This practice was founded on the supposition that it is quite possible that the beams may some time have to carry their full capacity in live loads, while the chances are increasingly less that the girders or columns will ever be required to carry anywhere near their full capacity, if a full load had been assumed. The fully loaded area would probably never be large, and a girder or column would rarely, if ever, lie in the centre of such an area. The effect of a live or moving load, causing vibration in the parts of the structure, is also gradually lessened as the vibration is taken up in the transfer of the load from member to member, so that by the time it reaches the footings or foundations the live load is ignored entirely. In fact, we can hardly imagine the perceptible effect on the foundations of the people in an office building, as compared with the infinitely greater *dead* load, due to the structure itself.

The former Chicago law required the floor-beams to be calculated for the entire assumed live load, while the girders could be taken as sustaining eight-tenths of the assumed live load plus the dead load, and the columns six-tenths of the live load plus dead load. This practice seems rational, and it was employed in much the greater proportion of Chicago's high buildings, but the revised or present ordinance prohibits such practice by requiring "the floors to be designed and constructed in such manner as to be capable of bearing *in all their*

parts, in addition to the weights of partitions and permanent fixtures and mechanisms that may be set upon the same, a live load of 100 lbs. per sq. ft."

A possible reduction in or omission of the live load on foundations is permitted, as follows: "In determining the areas of foundations for many-storied buildings, allowances are to be made for the fact that the before-mentioned live load is but an occasional load, which rarely occurs simultaneously upon corresponding parts of many floors, and if so, for a very brief period only."

In New York City, the previous building law required girders and columns to be calculated for the total live and dead loads, and also this total load to be assumed to rest upon the foundations. The present Building Code, adopted December, 1899, still requires the full floor-loads on girders, but provides for a reduction in the live loads on columns as follows:

"For the purpose of determining the carrying capacity of columns in dwellings, office buildings, stores, stables, and public buildings when over five stories in height, a reduction of the live loads shall be permissible as follows:

"For the roof and top floor the full live loads shall be used.

"For each succeeding lower floor it shall be permissible to reduce the live load by five per cent. until fifty per cent. of the live loads fixed by this section is reached, when such reduced loads shall be used for all remaining floors."

Building Laws: Live Loads on Floors.—For the purpose of comparison, the requirements of the Building Laws of New York, Chicago, Boston, and Philadelphia, for live loads per square foot of floor area, over and above the dead weight of the floor itself, may be classified as as on page 123.

Nearly all of these municipal requirements seem high when used by intelligent designers, except those for warehouses and manufacturing buildings. For dwellings, Kidder shows that *actual* loads in parlors (including piano), dining-rooms, etc.,

average only 14 to 23 lbs. per sq. ft. of the whole area. Data regarding experiments on the live loads in office buildings has already been given and with careful design and attention to detail, the writer believes that the requirements of most building laws are still too high. Loads for warehouses or manufacturing buildings are more difficult to calculate, and more difficult to enforce, for which reasons higher load units are to be expected and even desired than under more definite conditions.

	New York.	Chicago.	Boston.	Philadelphia.
1. Dwellings	60 (*a*)	40 (*e*)	50	70
2. Office Buildings	Upper floors 75 1st floor 150	100	100	——
3. Public Buildings	90	100	150 (*g*)	150
4. Stores, warehouses, factories, etc.	120 to 150 (*b*)	100 (*f*)	250 (*f*)	200 up (*f*)
5. Roofs	50 (*c*) 30 (*d*)	25	25 (*h*)	30
6. Sidewalks	300	——	——	——

(*a*) Includes apartment houses, tenements, and hotels.

(*b*) For ordinary stores, and light manufacturing or storage, not less than 120 lbs. For stores of heavy contents, warehouses and factories, not less than 150 lbs.

(*c*) For a pitch less than 20 degrees.

(*d*) For a pitch more than 20 degrees, measured on a horizontal plane.

(*e*) Includes hotels, boarding- and lodging-houses, and apartments.

(*fff*) Require posted notices of allowable loads.

(*g*) Except schoolhouses, which, except assembly rooms, require 80 lbs., and assembly rooms require 150 lbs.

(*h*) Additional allowance required for wind-pressure at 30 lbs. per sq. ft. No roofs, except dwellings, to have pitch greater than 20°.

The minimum load of 120 lbs. in the New York law is far too small in many cases, but the loads for warehouses, etc., are hard to classify, and are best left to the care of competent designers under the approval of the building departments. Mr. W. L. B. Jenney had occasion to estimate the loads in the wholesale warehouse of Marshall Field & Co. in Chicago, and

the surprisingly low average of 50 lbs. per sq. ft. was found for the total floor area, including all passageways. The maximum load on limited areas was found to be 57 lbs.

Dead Loads.—The dead loads to be considered in the floor-system include the arch itself, beams, concrete filling, floors (wood, marble, or mosaic), ceilings, and partitions.

The weights of the iron or steel beams and tile partitions are actually calculated for a typical floor plan, and then rated at so much per square foot of floor surface. This is absolutely necessary in regard to partitions in office buildings, as they are constantly being changed to suit the convenience of tenants. The weight of the arch varies with the depth; the depth is dependent on the span. In the annex of the Marshall Field Building, Chicago, the following weights were used:

Flooring, $\frac{7}{8}$-in. maple	4 lbs.
Deadening	9 "
15-in. tile arch	45 "
Iron	12 "
Plaster	5 "
Partitions, 3-in. mackolite	20 "
Total	95 lbs. dead load.

We have, therefore, for live and dead loads as follows:

	Beams.	Girders.	Columns.	Footings.
Offices: Live	85	65	45	....
Dead	95	95	95	95
Total	180	160	140	95
Store floors: Live	95	75	55	....
Dead	95	95	95	95
Total	190	170	150	95

The dead loads assumed in the Old Colony Building, Chicago (1893), comprised:

Flooring	4 lbs.
Deadening	18 "
Tile arches	35 "
Iron	10 "
Plaster	5 "
Partitions	18 "
Total	90 lbs.

The dead and live loads used in the calculations of the floor-systems and columns of this building were, in pounds per square foot:

	Beams.	Girders.	Columns.	Footings.
Live	70	50	40	—
Dead	90	90	90	90
Total	160	140	130	90

The floors for the Fort Dearborn Building were calculated in accordance with the following data:

	Dead Load.		Live Load.	
	Beams.	Girders.	Beams.	Girders.
1st floor	85	85	125	110
2d to 13th floors	75	75	70	60
Roof	40	40	40	40
Sidewalk	140	140	200	180
Prismatic lights	50	50	200	180
Skylight	....	....	40	40
Stairs	50	50	70	60

The live load on the beams from the second to thirteenth floor inclusive was taken at 70 lbs. per sq. ft., and an additional load of 20 lbs. per sq. ft. was added to the dead load to care for all partitions which were likely to be moved at any time.

The girders were figured for partition loads at 20 lbs. per sq. ft. for all movable partitions, and for the actual loads of the main partitions.

The live load on the columns was taken at 50 lbs. per sq. ft. from the second to the twelfth floor inclusive, plus the girder reactions for partitions.

The following table gives the unit loads used in figuring the columns:

	Live Load on Floor.	Live Load on Columns from Floors above.	Total Load on Columns.
Roof.......	40	....	....
13th floor...	50	40	40
12th " ...	....	45	85
11th " ...	....	41	126
10th " ...	....	35	161
9th " ...	....	31	192
8th " ...	....	25	217
7th " ...	....	21	238
6th " ...	....	15	253
5th " ...	....	11	264
4th " ...	....	5	269
3d " ...	....	1	270
2d " ...	....	0	270
1st " ...	125	0	270
Basement..	....	50	320

The dead load on the floor-beams was made up as follows, a 9-in. porous end-construction arch having been used:

9-in. arch	26 lbs. per sq. ft.
9-in. 21-lb. I-beams............	4 " " "
6 to 1 cinder concrete	30 " " "
Mosaic and wood floors, average..	10 " " "
Plaster	6 " " "
Total......................	76 lbs. per sq. ft.

In the Fisher Building, Chicago, 1895, the distribution of the loads for the roof, attic, and various floors was as follows.

	Load.	Joists. Lbs.	Girders Lbs.	Columns. Lbs.	Footings Lbs.
Roof	Live	20	15	15	
	Dead	40	40	40	40
	Total	60	55	55	40
Attic	Live	30	20	20	
	Dead	75	75	75	75
	Total	105	95	95	75
18th floor to 16th floor	Live	60	50	50	25
	Dead	75	75	75	75
	Total	135	125	125	100
15th floor to 13th floor	Live	60	50	45	25
	Dead	75	75	75	75
	Total	135	125	120	100
12th floor to 10th floor	Live	60	50	40	25
	Dead	75	75	75	75
	Total	135	125	115	100
9th floor to 7th floor	Live	60	50	35	25
	Dead	75	75	75	75
	Total	135	125	110	100
6th floor to 3d floor	Live	60	50	30	25
	Dead	75	75	75	75
	Total	135	125	105	100
2d floor	Live	75	60	40	25
	Dead	75	75	75	75
	Total	150	135	115	100
1st floor	Live	90	75	55	25
	Dead	75	75	75	75
	Total	165	150	130	100

N.B.—The weights of the fireproofing around the columns, and of the columns themselves, are added to the above column loads.

The dead loads in the same building were assumed as follows:

For floors,

$\frac{7}{8}$-in. maple flooring	4 lbs.
Cinder concrete deadening over floor arch	15 "
15-in. hollow-tile floor arch	41 "
Floor beams and girders	10 "
Plaster on ceiling	5 "
Total	75 lbs.

For roof,

3-in. terra-cotta book tile	22 lbs.
6-ply tar and gravel roof	6 "
T-irons to support terra-cotta book tile	4 "
Steel roof framing	8 "
Total	40 lbs.

Floor Framing.—Methods of floor framing, that is, the arrangements of columns, girders, and floor beams in skeleton structures, are illustrated in Figs. 58, 59, 60, and 61.

The first requisite in the design of the floor-system is the location of the columns. In a great measure the placing of the columns is governed by the arrangement of the exterior piers, the architectural effect striven for, or the arrangement and proper planning of the interior according to the intended uses. The column locations are thus usually the result of *conditions*, rather than any attempts at economy, but, unless complicated and expensive framing is to be expected, the distances between columns must always be kept within the limits of simple girder construction.

It is quite impracticable to make any comparisons as to the relative economy of many columns and short-span girders, and fewer columns with girders of longer span. Both types are to be found in practice, even to extremes, but the conditions governing the design of any particular building are usually so

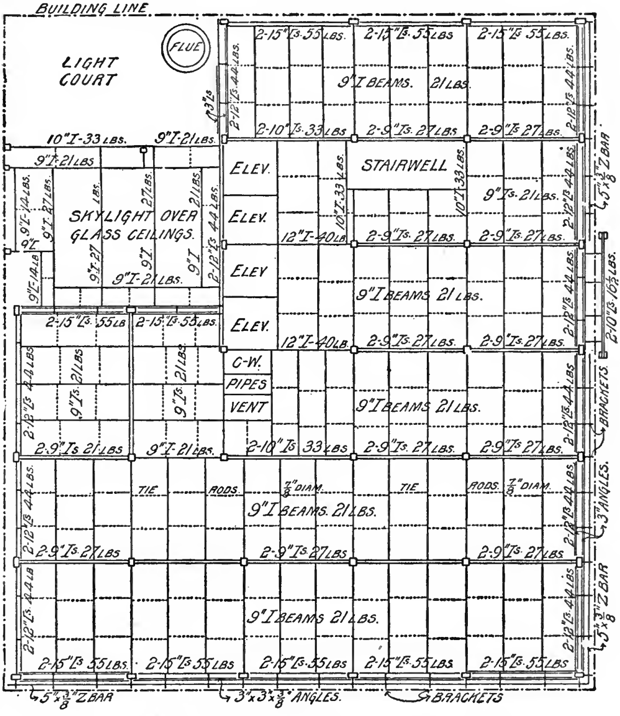


FIG. 58.—Typical Framing Plan, Fort Dearborn Building.

potent that a rule of column spacing in one instance would not be applicable in the next case.

For office buildings, the floor plans illustrated in Chapter III will indicate the arrangement of columns with reference to

office widths. The panels are usually made of such dimension as will give one wide office, or two suitable narrower

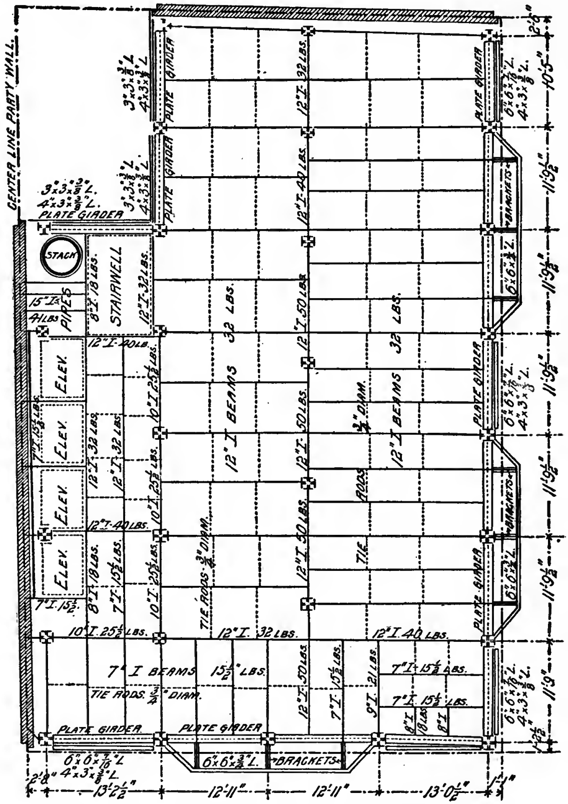


FIG. 59.—Typical Framing Plan, Reliance Building.

offices, from centre to centre of piers. Thus the practice of Messrs. Holabird & Roche, is to space both the exterior and interior columns 23 ft. centres, where possible, thus making

two offices of 11 ft. 6 ins. in each bay. See floor-plan of Champlain Building, Fig. 29.

The column centres having been determined, girders must next be located connecting the columns. The girders, running from column to column, in one direction at least, serve to support the floor-beams, transferring their loads directly to the columns. The girders also serve to brace the columns during erection, and they provide stability in the completed structure. Before the girders can be accurately calculated as to section, however, the floor-beams must be located, as the concentrated loads resulting from the beams determine the girder loads.

Floor-beams.—The spacing or distance centre to centre of the floor-beams will depend somewhat upon the type of fireproof flooring employed. The permissible spacing of beams for some of the more prominent fireproof floors has been given earlier in this chapter. For terra-cotta arches, which constitute by far the most general construction, ordinary practice in skeleton buildings has made 5 ft. to 6 ft. the most common span for panels of ordinary length. Where the columns are spaced a considerable distance apart, thereby causing long beams, the floor-beams are spaced nearer together—or not over 4 ft. to 4 ft. 6 ins. Reference to Figs. 58 and 59 will show the practice in beam spacing in two Chicago examples, while Figs. 60 and 61 illustrate framing plans for two prominent New York buildings.

The spacing of floor beams also depends upon the amount and character of the floor load, upon the length of span, and sometimes upon consideration as to permissible deflection. If the loads to be carried are largely static or motionless, as is usually the case, and if the span is small in comparison with the depth of the beam, the floor joists may be readily proportioned by means of the tables for "safe distributed loads" as given in the hand-books issued by the more prominent steel

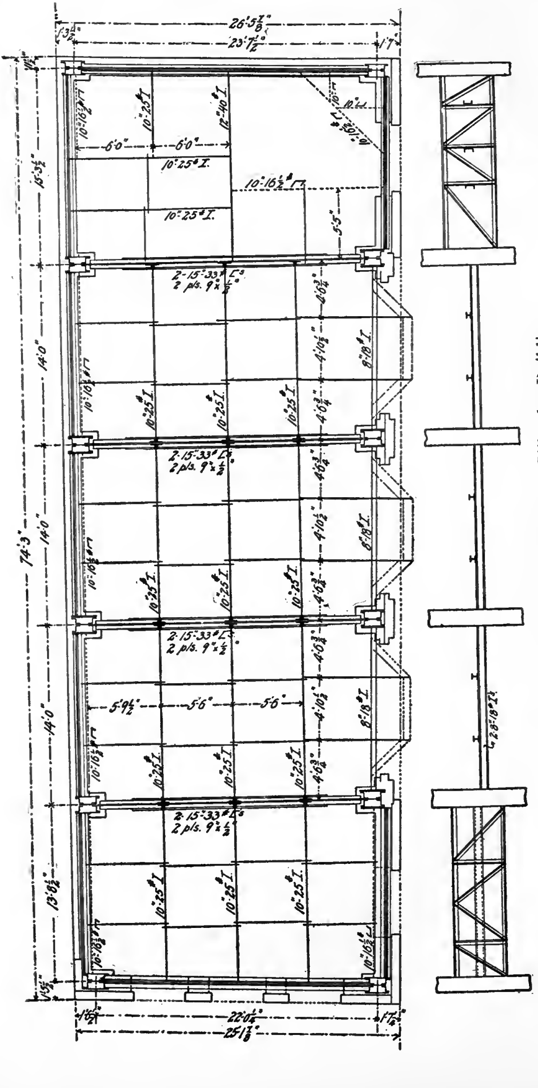


FIG. 60.—Typical Framing Plan, Gillender Building.

companies.* These tables give the loads, in tons of 2,000 lbs., which the various beams and channels will safely carry (distributed uniformly over the length) for distances between supports, as tabulated.

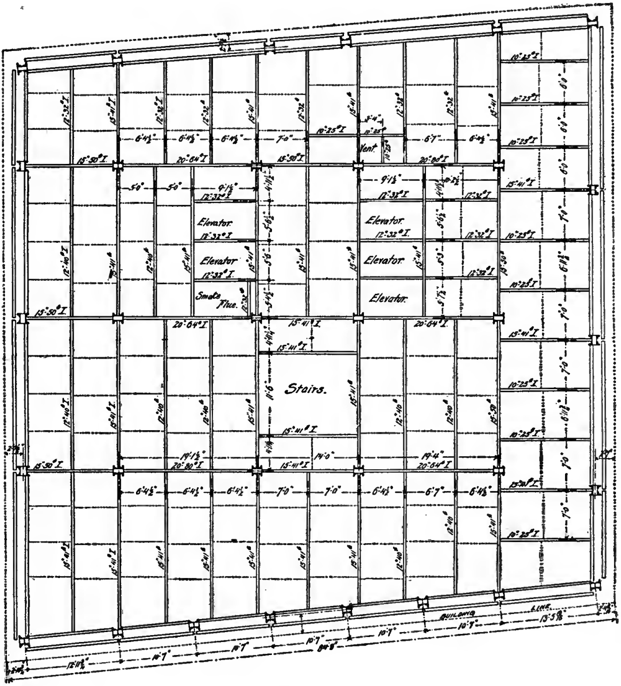


FIG. 61.—Typical Framing Plan, Am. Surety Co.'s Building, New York.

Or, if a section is to be selected to carry a certain load for a length of span already fixed, as is usually the case in build-

*For much valuable information and many useful tables concerning steel construction, the student is referred to the handbooks issued by The Carnegie Steel Co., Pittsburg, Pa., The Pencoyd Iron Works, Philadelphia, Pa., and others.

ing construction, the required beam or channel may be found by means of the coefficients given in tabular form for all rolled sections. For a uniformly-distributed load these coefficients are obtained by multiplying the load, in pounds uniformly distributed, by the span length in feet. If the load is concentrated at the centre of the span, multiply the load by 2, and then consider it as uniformly distributed. Such maximum coefficients of strength for I-beams and channels of different depths and weights per foot are given in the hand-books issued by The Carnegie Steel Company, L'd, and the Pencoyd Iron Works, the values being based on fibre strains of 16,000 lbs. per sq. in. (as used for ordinary loads), and 12,500 lbs. per sq. in. for rapidly moving or vibratory loads.

The Carnegie and Pencoyd hand-books also give tabulated values of the "Section Modulus" for all beams, channels, angles, Zs, Ts, etc., and in many respects the calculation of floor-joists, etc., by this method is to be preferred to the calculation by means of coefficients. The Section Modulus, wrongly called the Moment of Resistance, represents a constant property of any shape which may be considered as an index of the strength of such shape. The Section Modulus is constant for all spans and conditions of loading, as is also the weight per foot of the section considered, so that the former may be readily compared with the latter in determining the efficiency or economy of the section under consideration.

The Section Moduli are also very useful in determining the fibre stress per square inch for a beam or other shape subjected to bending or other transverse stresses. The Bending Moment in inch-pounds, divided by the Section Modulus, will give the extreme fibre stress to which the member is subjected.

Let S = Section Modulus, in inch-units;
M = Bending Moment, in inch-pounds;*

* For Bending Moments under various conditions of loading, see handbooks issued by the steel companies.

f = allowable stress per square inch in extreme fibres, usually taken at 16,000 lbs.;

W = total load in pounds, uniformly distributed;

l = length of span, in feet;

d = distance centre to centre of beams, in feet;

w = load per square foot in pounds;

Then

$$M = fS, \quad \text{or} \quad S = \frac{M}{f}.$$

But $M = \frac{Wl}{8}$ for a beam supported at both ends and uniformly loaded. Hence

$$S = \frac{Wl \times 12}{8f} = \frac{3Wl}{2f}. \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Also, as $W = dwl$,

$$S = \frac{3dwl^2}{2f}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Having found S from equation (1), the proper beam, channel, or other shape, may be selected from the tabulated values.

The most economical arrangement of floor-beams has had little investigation, and there seems to be no uniformity of practice. If the framing plans could be so arranged that the floor-beams and girders would be strained to the full allowable fibre strain, it would certainly be more economical than where the framing plans require the use of beams heavier than those actually needed. Take, for example, a framing plan calling for a bending moment in a floor-beam of 65,000 ft.-lbs. This would require a Section Modulus of 48.75. The Section Modulus for a 12-in. 40-lb. beam is only 44.8, while S for a 15-in. 42-lb. beam is 58.9. The latter would have to be used, with an excess in strength of some 20 per cent.; and if such panels occurred frequently in a floor system, an excess of 20 per cent. would therefore occur throughout. Hence an

economical framing plan would be one in which the beams are so arranged in span and distance centre to centre, as to carry a given floor-load with the beams strained to the full allowable fibre strain. A very small variation may make this possible or impossible.

If no beam can be found whose Section Modulus compares closely with the required value of S, it may be found practicable so to rearrange the spacing of the floor-joists as to permit of the economical use of some particular size of beam. In such cases, equation (2) may be solved for d, after substituting the desired value of S, thus obtaining the maximum spacing centre to centre, of the given I-beams.

Tables giving the maximum spacing, centre to centre of beams, will be found in both the Carnegie and Pencoyd handbooks for loads of 100, 125, 150, and 175 lbs. per sq. ft., for spans varying in length from 5 to 30 ft., and as the spacing of the beams is inversely proportional to the loads, the required spacing may be readily interpolated for loads other than those tabulated.

Also, in proportioning floor-beams, it is well to remember that it is seldom economical to use the heaviest weight of any depth of beam, if a deeper beam can be used. There is necessarily a great waste of material toward the ends of heavy rolled beams, and as the strength increases as the square of the depth, the deeper beam is always the more economical. Thus the Section Modulus for a 12-in. $31\frac{1}{2}$-lb. beam is 36, while for a 10-in. 40-lb. beam $S = 31.7$. The former is lighter, and far stronger. A 20-in. 65-lb. beam is also stronger than a 15-in. 80-lb. beam.

It will also be noted that, for the same depth of beam, the Section Moduli do not vary in proportion to the weight. The lightest weight of beam is invariably the most economical, providing its Section Modulus is slightly in excess of the value required by equation (1).

Care must be taken in figuring floor-beams to see that the length of clear span is not too great, giving a deflection sufficient to crack the plaster ceiling beneath. A deflection of about $\frac{1}{360}$ of the clear span, or $\frac{1}{30}$ of an inch per foot, has been found by experiment and practice to be the maximum permissible deflection, or $\delta = L \times 0.33$, where $\delta =$ greatest allowable deflection in inches, at centre of beam, and $L =$ length of span in feet. This safe deflection limit is also indicated for each size and weight of beam given in the tables for uniformly loaded I-beams in the mill handbooks.

Tie-rods.—With any arched form of fireproof flooring, whether flat or segmental, tie-rods are necessary in each bay to take up the arch thrusts without dependence on the adjoining arches. If all bays of the floor system were always loaded equally, tie-rods would be unnecessary, except in the outside panels; but with shifting live loads, tie-rods are almost invariably used, and are sometimes required by law. They are generally made $\frac{3}{4}$ in. in diameter, and spaced from 5 to 7 ft. apart. Intervals of eight times the depth of the floor-joists will about constitute average practice. Rods $\frac{7}{8}$ in. diameter are sometimes used in heavy work. Tie-rods are made with thread and nut on each end, to pass through open holes punched in the joists, usually at the centre of the beams. Some engineers specify that the holes shall be placed one-third the depth of the beam up from the bottom.

Girders.—The girders, running from column to column, support the floor-joists, and also the wall or spandrel loads when located between the exterior columns. The girders are usually deeper and heavier members than the regular floor-joists, being made of one I-beam, two I-beams side by side and connected by separators only, two I-beams with top and bottom riveted cover-plates, or lattice, plate, or box girders. Deep single beams, or lattice or plate girders are preferable. Closed sections, such as double I-beams or box girders, cause

inaccessible interior spaces which prevent future painting, and also require bolted connections through the webs. Separators should always be used in the case of double beams, unless connected by cover-plates, in order to equalize the load on the two beams, and also to act as spacers, keeping them a uniform distance centre to centre. Tables of standard-size separators are given in the mill handbooks.

Ordinary forms of girders applicable to building construction, other than single beams, are shown in Fig. 62.

FIG. 62.—Forms of Steel Girders used in Building Construction.

Tabulated coefficients of strength for these sections, for varying spans, are given in the handbooks before mentioned. Such coefficients of strength, or safe loads, are based on uniformly distributed loads, for fibre stresses of 13,000 to 15,000 lbs. per sq. in., but they may also be used for concentrated loads by following the directions given for use of tables.

Girders supporting floor-joists are usually calculated for concentrated loads instead of uniform loads, since the concentrated loads of the joists usually give smaller bending moments than would result from uniform loads over the tributary areas. Where a girder carries floor-joists on one side, and a floor arch on the other side, the member should be calculated for both cases of loading.

A point to be remembered in the design of girders is that a much more economical girder can be had when two floor-beams are to be supported than three; or an even number instead of an odd number of beams. In the latter instance a load will occur at or near the centre of the girder, resulting in a much greater bending moment. If but two beams are used,

the arm is but one-third the span of the girder. All of the floor-beams and girders in the floor system are usually so arranged as to be flush on the under sides, as shown in Fig. 63. This is to provide for the plastered ceiling. The inequalities in the arch depths are made up in the concrete filling.

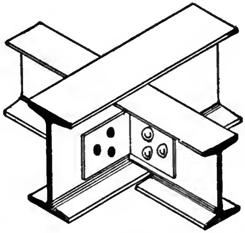


FIG. 63.—Isometrical View of Connection of Floor-beams to Girders.

If considerably deeper beams or lattice or plate girders are used for interior girders, the portions projecting below the ceiling line may be located on partition lines, and thus covered by plastered cornices, or false beams may be made to show in the rooms below, by means of fireproofing or metal furring and plaster.

In proportioning the sizes of floor-joists and girders, and columns as well, it is best to specify as few sizes of material and as few weights of beams and channels, or angles and zees, as may be practicable. Thus in the floor system, the calculations for the various spans and loadings may require a great number of beams and channels of different depths, and of different weights per foot for the same depths; but the inevitable delay in securing such a list of sizes and weights from the mills will often more than counterbalance the extra cost made necessary by using more uniform sizes and weights. Nearly all large orders from the rolling mills are rolled to order, and the sizes will be furnished as the rolling programme of the mill may permit. The lightest weights of the various sizes of beams and channels are almost always rolled first, while heavier or special weights may require a long delay before the orders of the mill make it desirable to roll such material.

Connections.—In buildings of moderate height, especially if constructed with solid masonry walls, bolted connections

may be used for almost all portions of the frame, but in veneer buildings of considerable height, riveted connections should invariably be specified.

For the connections of beams to beams, or beams to girders, connection-angles made after the standards adopted by the Carnegie Steel Co. or the Pencoyd Iron Works are almost universally employed on good work. The adoption of such uniform "standards" is certainly a great help to the mills and bridge or iron shops, as well as to the designer, but in the hands of the careless or ignorant designer is apt to be an element of weakness. From careful observation of building methods, as practised in general, the writer is convinced that faulty details constitute an even greater part of the defects in the general run of buildings, than arises from poor materials employed, or imperfect general features of design. Any "standards" are therefore to be used with caution, as they tempt the careless designer to use them under all conditions, whether they be adequate or not. They are *standard*, hence they *must* be all-sufficient.

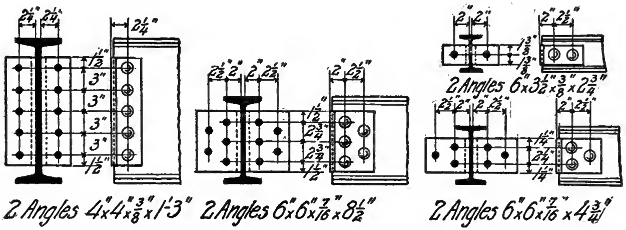
Standard connection-angles are designed on a basis of 10,000 lbs. per sq. in. allowable shearing strains, and 20,000 lbs. per sq. in. allowable bearing, and for regular details as found in ordinary practice these are *usually* of sufficient strength. But in extreme instances, where beams of short spans are loaded to their full capacity, it is often found necessary to provide additional strength in the connections. In such cases the limiting span lengths, or tables of minimum spans for fully loaded beams, must be followed. Typical standard connections are illustrated in Fig. 64, while Fig. 65 shows connections for beams of different depths framing into opposite sides of a girder.

On account of the difficulty of designing sufficient connections, no beam should frame into another one of less depth than itself, even though the calculated sizes would warrant it.

The additional cost of special connections in shop labor and erection will generally more than offset the additional weight required in a deeper beam.

In cases where it is impossible to make a sufficiently strong

5″ I and 6″ I.



18″ I and 20″ I. 12″ I. 8″ I and 9″ I.

Fig. 64.—Standard Beam Connections.

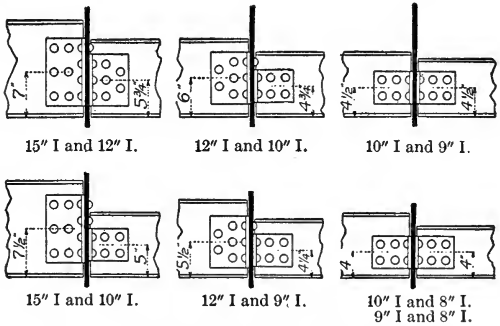


Fig. 65.—Connections for Beams of Different Depths.

connection through the web of a member only, a seat or shelf-angle may be riveted to the web of the girder immediately below the connecting member, to provide additional support, and, if necessary, this seat may in turn be reinforced by vertical stiffening angles.

In considering the connections of joists to girders, and especially girders to columns and columns to columns, in high buildings subject to considerable wind strains, Mr. Julius Baier lays especial emphasis on the following points, as the result of his investigations as to the behavior of building construction in the St. Louis tornado:*

"Well riveted joints in steelwork will stand, even under jar and shock, an excessive amount of abuse and distortion before actually separating into individual pieces.

"That for any twisting, wrenching, or bending strain, a $\frac{3}{4}$-in. rivet is far superior to the ordinary $\frac{3}{4}$-in. bolt.

"That the tension value of three $\frac{3}{4}$-in. steel rivets is sufficient to distort the web of a 15-in. 42-lb. I-beam $\frac{1}{2}$ in. out of line, without failure of the rivets, and is also far in excess of the bending resistance of the metal in a $\frac{7}{16}$-in. connection-angle.

"That an eccentric tension strain will readily cause a bolt to fail by bending or breaking in the thread, while the steel rivet will stand considerable distortion without failure."

Detailing.—It is comparatively seldom that complete detail plans for the steelwork of a building are made by the architect. Still less frequent are the cases where such detail plans could be used as actual shop drawings by the contractor, as in nearly every case the manufacturer much prefers to make his own shop drawings, to conform to the usage of his own plant. The architect has generally been content to specify the sizes and weights of the material to be used, leaving the details to be worked out by the contractor with the approval of the architect.

The experienced architect or engineer, however, is not usually satisfied with such license on the part of the contractor, and the best classes of work are made in accordance with

* See "Wind Pressure in the St. Louis Tornado, with Special Reference to the Necessity of Wind Bracing for High Buildings," by Julius Baier, Trans. Am. Soc. C. E., vol. xxxvii.

definite details furnished, after a careful consideration of the conditions to be fulfilled. This does not mean that complete shop drawings are made, but rather such connections and special points in the design as need particular attention. The balance of the detailing may be made to suit the contractor (with the approval of the architect), in conformity with the sizes of material marked on the plan, and the carefully drawn specifications.

The idea of allowing the manufacturer to prepare complete details after his own general scheme, and to follow specifications only, is not consistent with best results, in the judgment of the writer, though such an arrangement has often been advocated. It is true that it has been a very common practice with bridge engineers to furnish the moving-load diagram, and allow the bidders to design the structure as they saw fit, so long as it fulfilled all requirements of the specifications. This has probably been one reason for the high degree of excellence shown in the work of the better bridge companies, as each bidder endeavors to use his material to the best possible advantage. Such a practice, however, in building work will require a very careful supervision of the work by the architect, and as the various contractors will use those shapes most in favor, or of least cost, at their particular works, the calculations, connections, details, etc., must all be gone over and thoroughly checked, that all conditions may be satisfactory. A careful checking is necessary in any case, but where such complete freedom is accorded the bidder, it will rarely be that he is able to grasp the general ensemble in such a manner as to make satisfactory details in the required time. Again, only the most responsible and experienced firms could be intrusted with such a task.

Carefully drawn specifications, complete and accurate framing plans, sufficient spandrel sections and any special details, with all sizes and dimensions of material, will insure rapid and

satisfactory work on the part of the iron contractor. The shop drawings may then be examined, and stamped with the approval of the architect as received.

In detailing the floor system, the joists should frame into the webs of the girders in preference to resting on top of the girders. The latter method is cheaper, as the framing of the beams is avoided, but the former arrangement provides more stiffness and rigidity, besides avoiding the increased depth in the floors or the projection of the girders below the ceiling line.

Where the floor-joists are of the same depth as the girders, or where flush either top or bottom, "coping," or a cutting away of the ends of the flanges is necessary in the joists, to fit against the flanges of the girders. About $\frac{1}{8}$ in. clearance is usually allowed at each connection between floor-beams and girders, and $\frac{1}{4}$ in. between columns and girders. This is sufficient to overcome slight variations in the evenness of the material, and to permit of easy erection. Where beams or girders rest on column seats, the distances between the connection holes must be *exact*, with the allowances for clearance

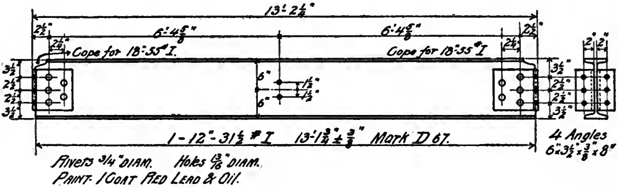


FIG. 66.—Shop Detail of Framed Beam.

made in the extreme ends between the end hole and the end of beam. Fig. 66 illustrates the shop detail for a framed beam. Fig. 67 shows the connections of beams, girders, and columns in "The Fair" Building, Chicago.

Beams and girders are usually numbered on the framing plans, to aid in detailing the various pieces, and to identify the parts for quick erection. The several floors or tiers are often

designated by letters, as "A" for first floor, "B" for second floor, etc. Columns are generally designated by one number for the entire height, while the various stories are given letters as in the beams. Thus, "Col. No. 2, tier 'B,'" would indicate a second-story column where the second floor was lettered "B."

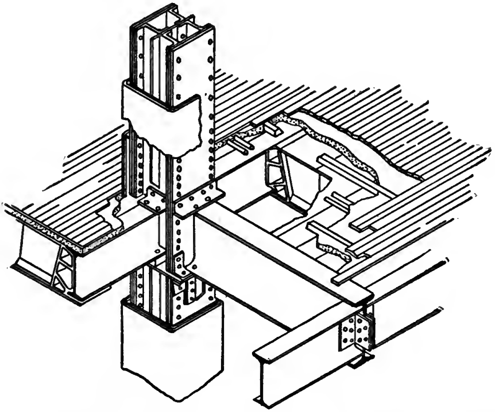


FIG. 67.—Connections of Beams, Girders, and Columns in "The Fair" Building, Chicago.

It is to be remembered that the cost of fabricating the material and the cost of erection are materially affected by the detailing employed. Strength and economy of manufacture and erection all require careful attention in detailing, as much may be saved or wasted in good or bad designing; and, as a structure is always gauged by its weakest point, so may an adequate amount of material be rendered insufficient through faulty minor details.

CHAPTER V.

EXTERIOR WALLS—PIERS.

A MOST striking example of the rapid and radical change which occurred in building practice as regards the construction of exterior walls, is shown in Fig. 16, Chapter III, where the old and new portions of the Monadnock Building, Chicago, are illustrated. The terms "old" and "new" are simply relative, as the newer addition was built some three or four years only after the original structure.

At the time of designing the older portion of this building, the owners, in spite of the protests of the architects, insisted on having the more conservative, and then eastern practice, of solid masonry piers, which, for the height of sixteen stories, resulted in walls six feet thick at the street-level. A few years later an addition was designed for the south half of the block, seventeen stories in height, and in this instance the walls were built after the veneer method, which had previously been rejected by the owners in the older portion. The difference in window areas and pier widths, especially near the sidewalk level, is apparent, even in the greatly fore-shortened illustration.

The exterior masonry walls for steel-frame buildings may be either "load-supporting" (as represented by the older portion of the Monadnock Building just mentioned), or "veneer-construction," *i.e.*, entirely dependent for support upon the steel frame (as in the newer portion of the Monadnock

Building), or "self-supporting," the latter being an expedient between the above-mentioned extreme cases.

Load-supporting Walls.—Load-supporting walls, built of solid masonry, and carrying all of the wall-, floor-, and roof-loads which come upon them without the use of steel or iron members, constitute the ordinary practice in buildings of moderate height, whether of fireproof or non-fireproof construction. Eight or ten stories is about a maximum height for load-supporting walls, so that in higher structures, which are here being considered in particular, it is a rare exception under modern methods to rely entirely on masonry piers.

The objections to such piers of solid masonry are threefold:

a. The modern requirements of plenty of light and air in all offices, demand that the windows be broad and numerous and the piers narrow. In the highest buildings of the present day hardly any masonry construction is strong enough to carry the necessary roof- and floor-loads besides its own weight, for so great a height and with so small a cross-section as is desired. There are prominent office buildings in almost all of our large cities, in which the exterior walls carry their proper share of all loads; but a little observation will show that in high buildings of this type the comfort of the tenants has, in a large measure, been sacrificed for architectural effect.

b. The second objection to such large masonry piers is that they take up too much valuable renting-space. When the rent of offices is proportioned at so much per square foot, this becomes a matter of no inconsiderable importance to the owner.

c. The weight of these solid masonry piers would so add to the load per square foot on compressible foundations that many of the most remarkable examples of architectural engineering would be well-nigh impossible.

In commercial buildings, of even considerable height, masonry piers are often used to carry all loads, but a mercan-

tile structure does not present as exacting conditions as an office building, and the exterior piers may be widened for architectural effect without seriously inconveniencing the plan of the interior.

A detail very common to store buildings is shown in Fig. 68. In such cases the first story especially is desired to have

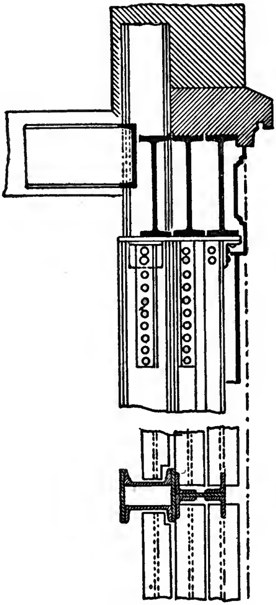


FIG. 68.—Detail showing Masonry Walls carried at Second Floor Level.

small piers and large windows for show purposes, in which case the solid masonry piers of the upper stories are supported at the level of the second or third floor on girders which are carried on steel columns running through the lower stories. In this illustration, the masonry walls are shown as carried at

the second-floor level, below which the girders and steel columns must be properly fireproofed, and covered with ornamental cast-iron fascias and column pilasters.

Self-supporting Walls.—Self-supporting masonry walls or piers are sometimes used, in which case additional metal columns, carrying the tributary floor- and roof-loads, are placed inside the masonry piers, while the latter support themselves and the "spandrels" only. The spandrels constitute those portions of the exterior walls lying between the piers and over and under the window-spaces.

If this method is employed, great care must be taken that the masonry does not touch the columns, in order that the *unequal* settlement of the metal-work and the masonry may not cause undesirable strains. On account of the numerous mortar-joints, the masonry will settle faster than will the metal columns under the gradual settlement of the whole structure. As an example of initial compression in freshly laid mortar, Mr. Geo. B. Post, architect of the New York Produce Exchange Building, states that a measured height of 9 ft. 6 ins. at the time of building, compressed about $\frac{1}{2}$ in. under a maximum pressure of 62 lbs. per sq. in. of base, induced by the finished wall. The whole wall was built very rapidly.

If, then, the masonry bears on rivet-heads, plates, or connections on the columns, a heavy strain is produced which has not been provided for. Great care is necessary in such combinations of metal columns and masonry piers to leave sufficient "open joints" at points over cornices and the like, where they will least be noticed, to allow for such settlement. Also where the mass is not homogeneous, as in stone facing and brick backing, the result is likely to be that the stone, with fewer mortar-joints, settles less and receives more than its share of the load, thus producing cracks and spalling off the angles.

This was the case in the old portion of the Washington Monument.

The objections of size and weight will also hold in the piers of this type, as in the first method, if the building be very high. Thus in the Masonic Temple of twenty-one stories, metal columns of plates and angles were placed within the masonry piers, but it was found that the maximum allowable pressure of 12 tons per square foot on brickwork, as was used in this instance, would be reached at the level of the fifth floor; hence below that level the load exceeded the safe compressive resistance of the material; and this without any floor- or roof-loads, as the latter were carried by the metal columns within the piers. The expedient was therefore adopted of carrying the masonry-work on brackets attached to the metal columns at the sixteenth- and fifth-story levels, thus making the pier consist of three separate columns of masonry, and the one continuous metal column.

As has been seen in Chapter I, self-supporting exterior walls were employed in most of the earlier examples of the so-called skeleton construction, the walls serving to carry their own weights, while all floor- and roof-loads were supported on metal columns placed within the walls.

The "World" Building, New York City, erected in 1890, is an extreme example of high building construction with self-sustaining walls. The main roof is 191 ft. above the street-level, making thirteen main stories, above which is a dome containing six stories,—in all, a height of 275 ft. above the street. The self-sustaining walls are built of sandstone, brick, and terra-cotta, the thickness increasing from 2 ft. at the top to as much as 11 ft. 4 ins. near the bottom, where the walls are offset to a concrete footing 15 ft. wide. The walls are vertical on the outside faces, the thickness being varied by inside offsets, so that the columns are recessed into the walls

at the bottom, but emerge and are some distance clear of the walls at the top.

Veneer-construction Walls.—The first example of a purely skeleton construction in Chicago occurred in the rear wall of the Phenix Building, now the Western Union Telegraph Building, by Burnham & Root, architects. In the wall behind the elevators, cast columns were used with two sets of horizontal supports at each story. The outside supports were made of I-beams resting on brackets connected to the columns, these I-beams carrying a $4\frac{1}{2}$-in. wall of enamelled brick. The inner supports consisted of I-beams placed between the columns, supporting a 4-in. wall of hollow tile. Thus the wall was formed of two layers or "skins" held together by the window-frames, etc.

But it was not until the introduction of the "cage-construction" steel frame, that veneer-construction walls and piers were fully developed; whereas now, with the general use of the independent steel framework, this type constitutes the most approved method—the one which has undoubtedly opened up the means for building the highest structures. In this, *all* weights are thrown on the metal columns, which, in place of solid piers, are surrounded with a protective shell or covering only, made of ornamental terra-cotta or brickwork, securely anchored to, and supported by the columns at the various floor-levels.

This construction undoubtedly gives the minimum weight per foot of height, and makes possible such small piers as are indispensable for light and desirable offices. The "Chicago type" is a popular name for this method; a type which has developed very remarkably during the past few years of American architecture, while the height of municipal buildings has been increasing steadily from ten to thirty stories. The increasing value of ground-space, the demands for rapid con-

struction, and the necessity for the lightest possible loads on the subsoil, have all contributed to the success of this detail.

Veneer construction thus does away with masonry as a supporting member, and the load-bearing brick wall or masonry pier is replaced by an envelope of terra-cotta or brickwork, enclosing the steel columns and filling the spandrels or spaces between the windows. This envelope is not used as a strengthener to the supporting members, but as a protection against the elements and the dangers of fire. The brick wall, once the fundamental factor in building construction, now fulfils simply a decorative and protective function. The great possibility for external effect through this use of brick and terra-cotta in connection with skeleton construction, has opened up a vast market to the manufacturers of fine qualities of face-brick, moulded brick, and terra-cotta in all its varieties.

The terra-cotta companies design their pieces with especial reference to tying them to, or suspending them from such a framework; so that, in reality, the building becomes nothing more nor less than a vital skeleton of steel, with an architectural and protective wrapper of terra-cotta, tile, or brickwork, inside and outside. The terra-cotta arches, which to the casual observer seem to carry some heavy wall or pier above, prove to be made of hollow clay blocks, held by clamps to the concealed beams or girders which really support the loads.

Materials used in Exterior Walls.—In the construction of exterior walls, piers, and spandrels (see Chapter VI for especial reference to spandrels), the selection of methods and materials must be made with a view to fulfilling, as far as may be possible, the requirements as to adaptability of form and facility of handling, the protection of the steel frame against corrosion and deteriorating influences, and protection against damage by severe fire and water tests, either from internal or external sources.

In general, it may be stated that brick and terra-cotta are

generally preferred to other building materials for the exterior walls of high buildings, on account of the ease with which they may be handled, the facility with which they may be built into and about the forms of columns and beams, as well as on account of their superior fire-resisting qualities. For more detailed information as to the materials of fireproof construction, the reader is referred to Chapter V, Materials used in Fire-resisting Construction, in the author's "Fireproofing of Steel Buildings."

Fire-resisting Qualities.—As regards protection against internal or external fire hazard, the selection of proper fire- and water-resisting materials is of the utmost importance in the construction of walls or piers. If the aim is simply to secure *incombustible* materials, almost any form or character of metal-work or masonry may be employed—presuming, of course, that the load-carrying steel frame is properly protected against possible injury, regardless of the character of the decorative facings. But such construction is very liable to prove a poor investment, as may be shown by numerous examples of notable fires in which large portions of so-called fireproof buildings have required reconstruction to such an extent that the losses occasioned through the use of injudicious materials have proved quite the most considerable items involved. This experience has been particularly true of stone, where the material is non-combustible, and hence adds no fuel to the conflagration, but where the inevitable destruction under severe test conditions often makes reconstruction both difficult and expensive.

Brick masonry and terra-cotta, however, are unsurpassed as fire-resisting materials, as has been amply proven by innumerable conflagrations. These products have stood repeated fire and water tests of great severity and considerable duration, where all other ordinary building materials have suffered complete destruction or at least extensive damage. The endurance of brickwork was fully demonstrated in both

the Pittsburg and Home Insurance Building fires, where face-brick suffered but little damage except through discoloration. In the latter fire, the excellent qualities of architectural terra-cotta were also clearly shown.

Stone Masonry.—Marble, limestone, or granite should never be relied upon as forming protection against fire, or to carry loads other than their own weight. If used at all, they should be employed in such manner that the strength of the structure is in nowise dependent upon their use; and even then, from the consideration of reconstruction, the more limited the use, the better. Four-inch or five-inch slabs, such as are frequently found in veneer construction, form very little protection against fire, and even where such facing slabs are made of greater thickness, they should be backed up with sufficient brickwork or terra-cotta to insure the full protection of the steelwork in case the stone veneer is destroyed. Furthermore, the backing or true fireproofing should be independent of the facing for support, so that the destruction of the latter would not cause the failure of the fireproofing.

The financial loss due to the use of limestone was well shown in the Chicago Athletic Club Building fire; the danger to life from dropping stone during fire and the practically complete destruction of marble was illustrated in the Home Life Building fire; the large Boston fire on Bedford Street showed the injurious effects of fire and water on brown sandstone, while examples too numerous to mention might be cited as to the utter unreliability of granite.

Stone has not been extensively employed in skeleton construction, except in the lower stories only, as a base for the superimposed brick or terra-cotta work, or in conspicuous exceptions where used throughout. This has been largely due to the difficulty experienced in properly attaching the stonework to the metal framework, as well as to considerations of fire resistance previously mentioned. In some instances, stone

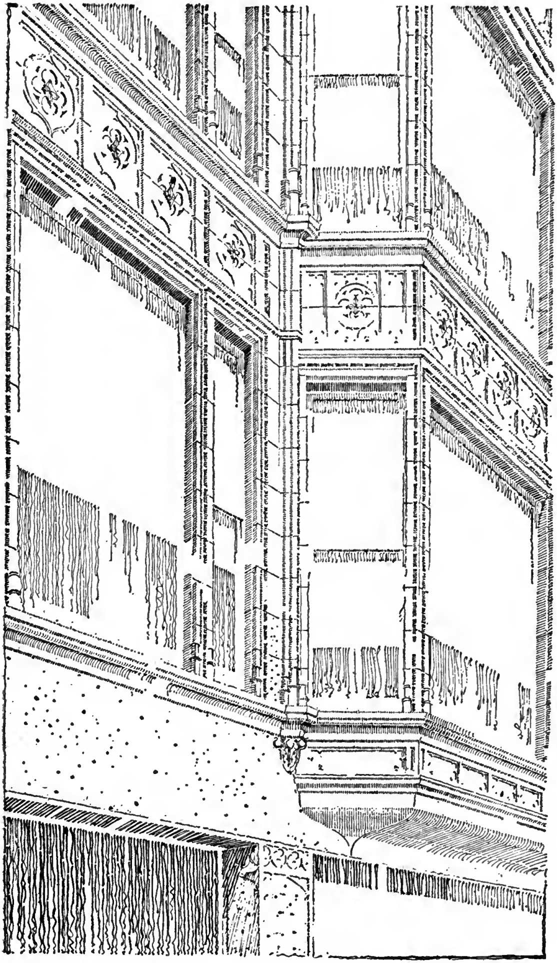


FIG. 69.—Detail of Terra-cotta Front. Reliance Building.

has been used in thin slabs in the lower stories, as in the first floor of the Reliance Building, where highly polished slabs of granite were enclosed within ornamental frames or grilles of

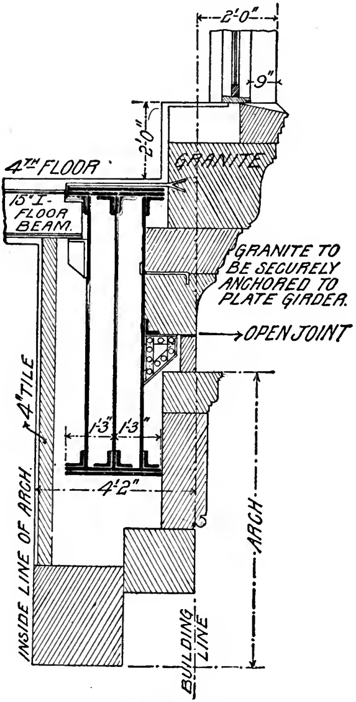


FIG. 70.—Section through Wall at Main Entrance to Masonic Temple.

cast-iron, over the fireproofing, as shown in the lower part of Fig. 69.

An example of the attachment of stone masonry to the steel frame is shown in Fig. 70, where the box girder over the main entrance of the Chicago Masonic Temple is illustrated in

connection with the granite arch which extends up into the third story, as shown in previous Fig. 15. The box girder supports two steel columns within the piers over the entrance, the masonry piers themselves and their share of the spandrel loads, besides the fourth-floor beams and the granite-work above the arches shown.

Brick and Terra-cotta.—It has already been said that both brick and terra-cotta possess, in a remarkable degree, great advantages as to erection. This is due to the ease with which they may be handled, as well as to their ready adaptation of form. Terra-cotta is easily moulded into almost any required shape, thus providing for a suitable attachment to the steel frame; it is susceptible of elaborate ornamentation or modelling; it may be obtained in a wide range of colors and finishes; and it may be used to produce a great variety of architectural effects, either separately or in combination with brickwork. These considerations, in addition to most admirable fire-resisting qualities, all contribute to the success which has attended the wide use of terra-cotta. The rich decorative possibilities which brick and terra-cotta possess, are well illustrated in the new Broadway Chambers Building, New York.

Method of Setting.—The terra-cotta blocks, as used in exterior wall construction, are usually built up in advance of the brick backing, one course at a time. They should always be backed up with brick masonry, or with structural terra-cotta as is sometimes employed. The voids in the rear of the face blocks should always be filled, where possible, with bricks or parts of bricks, well filled in with mortar, to make the construction as firm as possible. A thickness of 8 ins. for external terra-cotta and backing should be taken as a minimum.

After setting, all joints in terra-cotta work should be well raked out to a depth of $\frac{3}{4}$ in., and be "pointed" with Portland cement mortar, colored to suit the architectural effect required.

Hooks, Ties, etc.—The individual terra-cotta blocks should be anchored to the backing, or directly to the steelwork, by means of anchors or hooks made of galvanized-iron, or iron dipped in coal-tar or graphite paint. Methods of anchoring are shown in more detail in Chapter VI in connection with spandrel sections.

The brick backing should also be anchored to the steel-frame, either by hooking anchors over members of the frame, or by passing them through open holes provided in the beams, columns, etc., for that purpose.

Wall Columns.—The earlier method of surrounding exterior columns by masonry piers is shown in Fig. 71. The

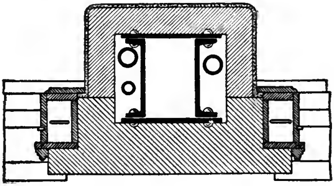


FIG. 71.—Fireproofing of Columns in Exterior Walls.

cast-iron or steel columns were placed within the walls in such manner as to leave from 4 ins. to 12 ins. of masonry between the columns and the exterior face of the wall, with the balance of the columns projecting into the room areas, where terra-cotta fireproofing was placed around the thus exposed portions. Not infrequently, drainage, water, or even steam-piping was run alongside the metal columns, and within the fireproof coverings.

A later and better example is shown in Fig. 72, which illustrates a corner pier in the Reliance Building, Chicago, 1894. Fig. 73 is a plan of the supporting framework for the same corner, showing the shelf-angles on the column for the support of the pier, and the plate girder and spandrel angles for carrying the spandrel portions of the walls between the

piers. These two figures also show the cast-iron uprights employed to stiffen and secure the terra-cotta mullions.

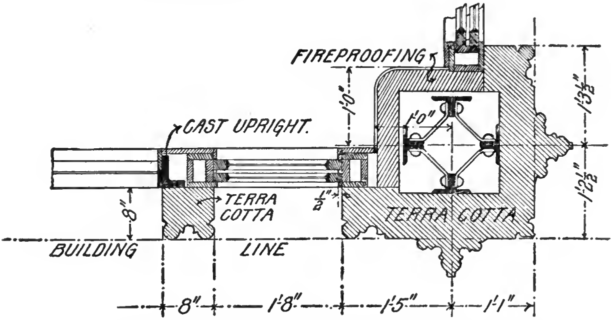


FIG. 72.—Detail of Corner Pier and Column. Reliance Building.

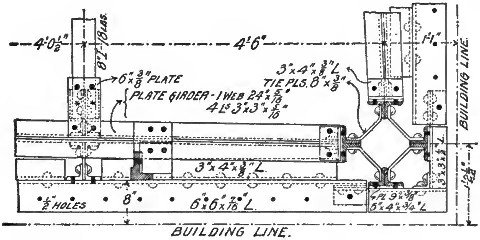


FIG. 73.—Detail of Wall Girders and Corner Column. Reliance Building.

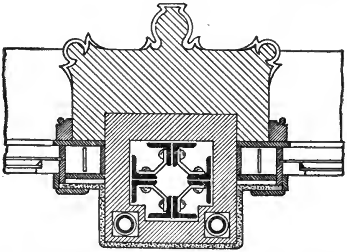


FIG. 74.—Detail of Columns in Exterior Walls. Fisher Building.

In still later and better examples of exterior piers, the fireproofing is made to surround the steel columns completely, so

that the brick or ornamental terra-cotta front is not relied upon as the only external protection. See Fig. 74. This illustration also shows the most approved method of caring for all piping within slots or recesses provided in the fireproofing, these being separated from the metal members by a thickness of terra-cotta, or wire lath and plaster sufficient to prevent corrosion or deterioration from changes in temperature, moisture, or deleterious gases, etc.

"Free-standing" Wall Columns.—A special detail of exterior piers and columns has been developed by Architect Geo. B. Post, and used by him in the St. Paul Building, New York City. In this building, all of the exterior columns are located entirely within the interior face of the brickwork, thus standing free within the rooms. This arrangement is shown in Fig. 75. The outside flanges of the columns are placed not less than 16 ins. from the outside of the masonry. The piers are carried by horizontal plates and wall beams, which are in turn carried by cantilevers formed by projecting the members of the floor system out beyond the column. For this purpose the floor girders are made double, so that one member may pass the column on either side. Knee-braces, made of gusset-plates and angles, are riveted to the column above and below the cantilevers, thus providing rigid bracing.

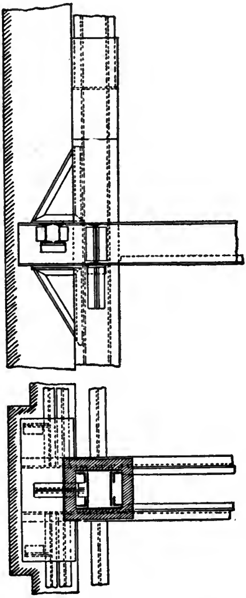


Fig. 75. — Detail of "Free-standing" Wall Columns. St. Paul Building.

Before the building of the masonry walls, the columns were encased with porous terra-cotta 4 ins. thick, and between this casing and the masonry a further protection against corrosion was introduced in the form of asphalted felt, laid to form a damp-proof course on the sides of the column next to the brickwork. A space left between the asphalt sheet and the masonry was later filled with a grouting of cement mortar.

This design was intended to secure, first, improved exclusion of moisture and prevention of corrosion; second, superior fireproofing; third, accessibility for inspection or repairs if necessary; and fourth, connections of the floor system and pier loads so as to avoid the eccentric loading of the columns.

Protection of Exterior Metal-work.—The protection of the steel frame against corrosion, deterioration, etc., was discussed in Chapter III, but in considering exterior walls and spandrels the fact must be borne in mind that, while less is now required of the brick or masonry wall as a supporting member than formerly, when the walls fulfilled the function of bearing dead-loads, much more is now demanded of it as to quality and perfection of workmanship, in order that adequate protection may be afforded the vital steel frame within.

In order to render the exterior impervious to moisture, and thus protect the metal framing against corrosion, brick masonry, whether employed as a backing for other materials or as finished brickwork, should be built of the best possible materials. Only the very hardest and most thoroughly burned brick should be used, and cement mortar is generally specified in the best classes of work, with well-filled joints and careful bonding and anchoring. Cement mortar is especially important where the mortar comes in contact with the steelwork.

A thickness of 4 ins., or a single brick, is often used for external protection, but a minimum of 8 ins. is greatly to be preferred for efficient security against fire and corrosive influences.

Protection of External Members; Building Laws.—The Chicago Building Ordinance requires the following for the protection of external structural members: "All iron or steel used as a supporting member of the external construction of any building exceeding 90 ft. in height shall be protected as against the effects of external changes of temperature and of fire by a covering of brick, terra-cotta, or fire-clay tile, completely enveloping said structural members of iron and steel. If of brick, it shall be not less than 8 ins. thick. If of hollow tile, it shall be not less than 8 ins. thick, and there shall be at least two sets of air-spaces between the iron and steel members and the outside of the hollow-tile covering. In all cases the brick or hollow tile shall be bedded in mortar close up to the iron or steel members, and all joints shall be made full and solid.

"Wherever stone facing is used, it shall be an additional thickness to the column covering above specified.

"Where skeleton construction is used for the whole or part of a building, these enveloping materials shall be independently supported on the skeleton frame for each individual story.

"If terra-cotta is used as part of such fireproof enclosure, it shall be backed up with brick or hollow tile; whichever is used being, however, of such dimensions and laid up in such manner that the backing will be built into the cavities of the terra-cotta in such manner as to secure perfect bond between the terra-cotta facing and its backing."

The New York law prescribes the following: "Where columns are used to support iron or steel girders carrying enclosure walls, the said columns shall be of cast-iron, wrought-iron or rolled steel, and on their exposed outer and inner surfaces be constructed to resist fire by having a casing of brickwork not less than 8 ins. in thickness on the outer surfaces, not less than 4 ins. in thickness on the inner surfaces, and all bonded into the brickwork of the enclosure walls.

"The exposed sides of the iron or steel girders shall be

similarly covered in with brickwork not less than 4 ins. in thickness on the outer surfaces and tied and bonded, but the extreme outer edge of the flanges of beams, or plates or angles connected to the beams, may project to within 2 ins. of the outside surface of the brick casing.

"The inside surfaces of girders may be similarly covered with brickwork, or if projecting inside of the wall, they shall be protected by terra-cotta, concrete, or other fireproof material.

"Girders for the support of the enclosure walls shall be placed at the floor line of each story."

Protection of Column Interiors.—If the steel columns employed are of a box section, or closed form, as is often the case in such types as Z-bar columns with cover-plates, or plates and angles in rectangular form, a further protection against corrosion may be obtained by filling the column interiors with rich Portland cement concrete. Closed columns naturally do not permit of finished painting after the fabrication of the members, nor of inspection nor renewal of painting at later dates. Columns in exterior walls or other exposed locations are, therefore, sometimes lined with Portland cement, or filled with cement concrete as a permanent precaution against possible deterioration. All of the exterior columns in the Ellicott Square Building, Buffalo, N. Y., were thus filled with Portland cement concrete.

Anchorage.—The question of proper anchorage of the brickwork and terra-cotta to the steel frame has been mentioned before, but this point is worthy of especial emphasis. This subject is often entirely overlooked in writing specifications, but in all classes of work, of whatever character, adequate anchorage is very important. In load-supporting walls, efficient bracing must be obtained by means of connections to the interior frame, and proper anchorage will often prevent the collapse of the walls from hot-air explosions, etc. In veneer

construction, also, the comparatively thin curtain walls must largely rely for stability on their anchorage to the steelwork, as well as upon their inherent strength. Speaking of experience gained through the St. Louis tornado, Mr. Baier states that "the great amount of explosive action was largely due to the comparative weakness of ordinary walls against pressure exerted from the inside of buildings. A more efficient anchorage of the walls might limit this explosive action to the windows. In numerous instances the windows were blown in on the windward side, while the entire walls were blown out on the leeward side. Brick walls are materially stronger if well bonded with the vertical joints filled with mortar, and a wall laid in cement will undoubtedly withstand a greater lateral force than one laid in lime mortar." *

Party Walls.—Columns or beams located within party walls should always be efficiently protected by their own masonry, without reference to the walls of adjoining buildings. In many cases the steel columns and wall-beams for large and important new structures have been placed directly against the walls of neighboring buildings, which, in case of fire, are apt to suffer complete destruction, thus exposing the steel members of the newer building.

Thickness of Walls.—As regards the thickness of walls required, for whatever class of building, this is generally specified by the local building ordinances. There is considerable variance, however, in the requirements for veneer walls in cage construction.

A brick wall carried to the height of the Manhattan Life Insurance Building in New York City (241 ft.) would, according to the building laws of most cities, have to be about 6 ft. thick. Through the use of skeleton construction the enclosing walls in this building were made only 12 and 16 ins. thick.

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* See Julius Baier in Transactions Am. Soc. C. E., vol. xxxvii.

Fig. 76 shows the required thickness of walls under the Chicago ordinance for buildings devoted to the sale, storage, and manufacture of merchandise. Fig. 77 is for the walls of

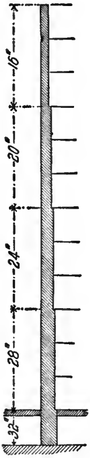

Fig. 76.—Diagram of Wall Thicknesses for Mercantile Buildings, Chicago Ordinance.

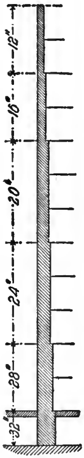

Fig. 77.—Diagram of Wall Thicknesses for Other than Skeleton Construction.

hotels, apartments, and office buildings, of construction other than the skeleton type.

These thicknesses, in Figs. 76 and 77, are for the maximum allowable height of 130 ft. from the sidewalk level to the highest point of external walls.

For skeleton construction, the Chicago ordinance allows veneer walls of 12 ins. thickness for any height within the maximum limit of building height above stated. The New York City building law requires the use of 12-in. curtain walls for 75 ft. of the uppermost height thereof, and 4 ins. additional

thickness for every lower 60-ft. section down to the sidewalk level. But, on account of the severity of these requirements as applied to very high cage-construction buildings, permission is frequently given by the Board of Examiners, who are empowered to modify the building laws within certain limits, to reduce the above-mentioned thicknesses to 12 ins. and 16 ins. for buildings greatly exceeding 100 ft. in height. They have never, however, permitted a uniform thickness of 12 ins. for buildings over twelve stories in height.

Allowable Unit-stresses. — The allowable pressure per square foot on brick masonry, as used in the highest masonry piers in Chicago, namely, in the Masonic Temple, has been mentioned before as 12 tons.

Prof. I. O. Baker, in his "Treatise on Masonry Construction," gives the following allowable strains on brickwork as the practice of the leading architects:

10 tons per sq. ft. on best brickwork laid in 1 to 2 Portland cement mortar;

8 tons per sq. ft. for good brick laid in 1 to 2 Rosendale cement mortar;

5 tons per sq. ft. for ordinary brick, laid in lime mortar.

He shows, however, that these figures are very conservative, as his tables of the ultimate strength of best brickwork give from 110 tons with lime mortar to 180 tons with Portland cement mortar per square foot. So while the unit of 12 tons in the Masonic Temple was even greater than ordinary Chicago practice, Prof. Baker adds that "reasonably good brick laid in lime mortar should be safe under a pressure of 20 tons per sq. ft."

The safe loads given in the Boston law are about double those recommended by Prof. Baker, while the New York requirements, using $\frac{1}{10}$ of the average ultimate strengths given by Prof. Baker, allow 114 tons on granite, 90 tons on limestone, and 72 tons on sandstone, per square foot.

BRICKWORK: ALLOWABLE PRESSURES IN TONS PER SQUARE FOOT, SPECIFIED BY BUILDING LAWS.

	New York.	Chicago.	Boston.
Brickwork laid in cement mortar	15 (a)	12½ tons with Portland cement.	15 (b)
Brickwork laid in cement and lime mortar	11½ (a)	9 tons with ordinary cement.	12 (b)
Brickwork laid in lime mortar	8 (a)	6½ tons with lime mortar.	8 (b)

(*a*) Isolated brick piers shall not exceed 12 times their least dimensions.

(*b*) In brick piers in which the height is from 6 to 12 times the least dimension, these pressures are reduced to 13, 10, and 7 tons respectively for the mortars as above given.

STONE MASONRY: ALLOWABLE PRESSURES IN TONS PER SQUARE FOOT, SPECIFIED BY BUILDING LAWS.

	New York.	Chicago.	Boston.
Granite	$\frac{1}{10}$ of the ultimate strength.	Not specified.	60 First quality, dressed beds, laid solid in cement mortar.
Marble and limestone	$\frac{1}{10}$ of the ultimate strength.	Not specified.	40 First quality, dressed beds, laid solid in cement mortar.
Sandstone	$\frac{1}{10}$ of the ultimate strength.	Not specified.	30 First quality, dressed beds, laid solid in cement mortar.

The use of ashlar masonry in wall facings is limited as follows: Boston law: "In reckoning the thickness of walls, ashlar shall not be included unless it be at least 8 ins. thick. In walls required to be 16 ins. thick or over, the full thickness of the ashlar shall be allowed; in walls less than 16 ins. thick, only half the thickness of the ashlar shall be included. Ashlar shall be at least 4 ins. thick, and properly held by metal clamps to the backing, or properly bonded to the same."

Chicago law: "Stone may be used as facing for brick walls under the following conditions: If the facing is ashlar, without bond courses, and the individual courses thereof measure in height between bond-stones more than six times the thickness of the ashlar, then each piece of ashlar facing shall be united to the brickwork with iron anchors, at least two to each piece,

and reaching at least 8 ins. over the brick wall, and hooked into the stone facing as well as the brick backing. Wherever ashlar, as before described, is used, it shall not be counted as forming part of the bearing-surface of the wall, and the brick backing shall be of the thickness of wall herein specified for the different kinds of building.

"If stone facing is used with bond courses at a distance apart of not more than four times the thickness of the ashlar, and where the width of bearing of the bond courses upon the backing of such ashlar is at least twice the thickness of the ashlar, and in no case less than 8 ins., then such ashlar facing shall be counted as forming part of the wall, and the total thickness of wall and facing shall not be required to be more than herein specified for walls of the different classes of buildings."

New York law: "All stone used for the facing of any building, and known as ashlar, shall not be less than 4 ins. thick. Stone ashlar shall be anchored to the backing, and the backing shall be of such thickness as to make the walls (independent of the ashlar) conform, as to the thickness, with the requirements of this ordinance."

CHAPTER VI.

SPANDRELS AND SPANDREL SECTIONS—BAY WINDOWS.

THE spandrels constitute those portions of the exterior walls, either on the street fronts or on interior courts, which lie between the piers and between the window-spaces of successive stories. "Spandrel sections," as they are called, must be made for every different type of spandrel support in the building, and they must clearly show the supporting beams or metal-work required to carry the veneer walls in the manner desired. These sections vary greatly, depending largely on the architectural effect contemplated by the designer in his arrangement of the material, and general descriptions of spandrels can hardly be given as applicable to general practice. Illustrations of numerous examples will better serve to show the methods employed.

The spandrel-beams are supported by the masonry piers where such load-bearing piers are used, or, in the veneer construction, by the metal columns in the walls. The face of the spandrel-walls may be "flush" with the piers, or "in reveal," that is, set back from the face of the piers. In the first case the wall presents a nearly unbroken surface, except for the terra-cotta sills and window-caps, while the second method accentuates the piers, and throws the spandrel-walls in reveal. The architectural treatment will determine these conditions. The former case is generally of far simpler construction, as the spandrel-beams come at or near the centres of the columns, thus avoiding many embarrassments in the irregular bracketing from the columns, which becomes necessary in the support of

the spandrel-beams where the spandrel- or curtain-walls are recessed.

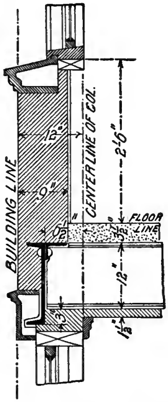

FIG. 78.—Spandrel Section. Ashland Block, Chicago.

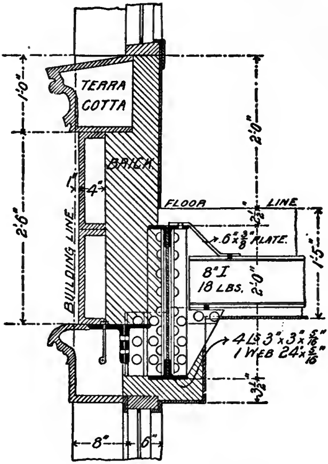

FIG. 79.—Spandrel Section. Reliance Building.

Fig. 78 shows a very simple form of spandrel section from the Ashland Block, Chicago, where flush walls were used. The veneer wall is but 9 ins. thick.

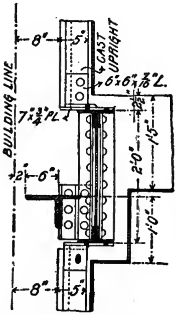

FIG. 80.—Connection of Cast Mullions. Reliance Building.

The use of plate girders, as the main spandrel supports, is shown in Fig. 79, which is a section taken from near the corner of the Reliance Building. The connections of these plate girders to the Gray columns used are shown in Fig. 118, Chapter VII. The connections of the cast uprights which support the terra-cotta mullions between the windows are shown in Fig. 80. Compare with Figs. 72 and 73. Figs. 81 and 82 are taken from the eleventh- and twelfth-

floor levels respectively of the Fort Dearborn Building. The section given in Fig. 83 is taken at the first-floor or sidewalk level, and shows the prismatic lights in the sidewalk, as well

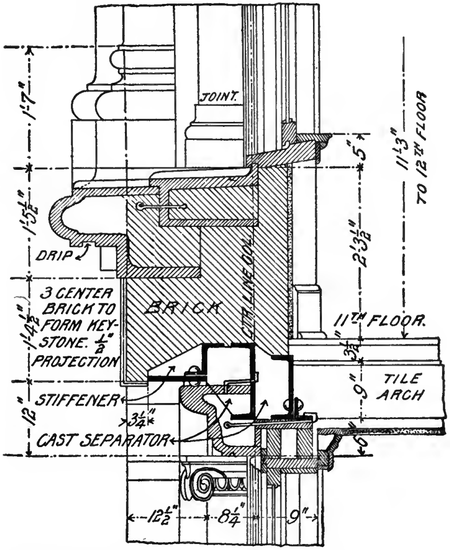

FIG. 81.—Spandrel Section, Eleventh Floor. Fort Dearborn Building.

as the small windows which help to light the basement restaurant space. Fig. 84 is a section taken at the attic floor, showing the main cornice and roof construction.

The materials generally used for veneer buildings consist, as before stated, of pressed brick and terra-cotta, the latter being used for the window-caps and sills, horizontal bands, ornamental capitals, brackets, etc., or even in entire façades, according to the architectural treatment desired.

The brick or tile work of the piers is usually supported by bracket-angles, attached to the columns, as has been described

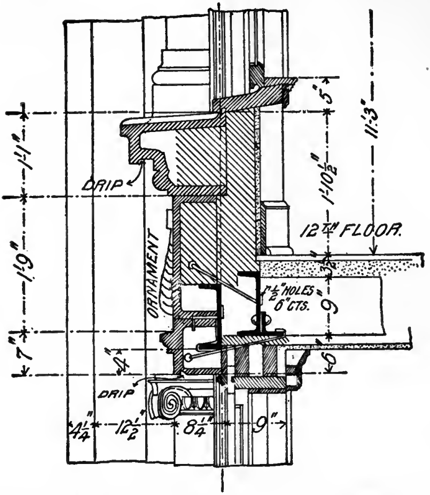

FIG. 82.—Spandrel Section, Twelfth Floor. Fort Dearborn Building.

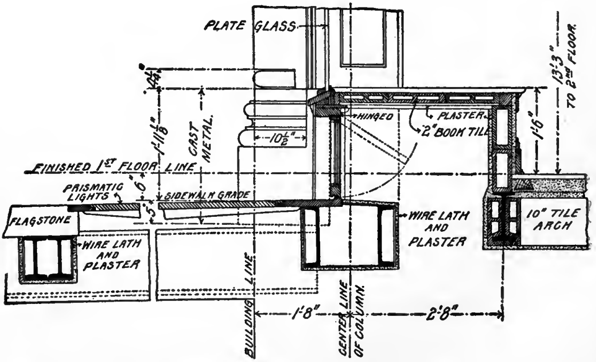

FIG. 83.—Spandrel Section. First Floor. Fort Dearborn Building.

in Chapter V, while the body or backing of the spandrel-walls is supported directly by the main spandrel-beams, as indicated in the previous figures.

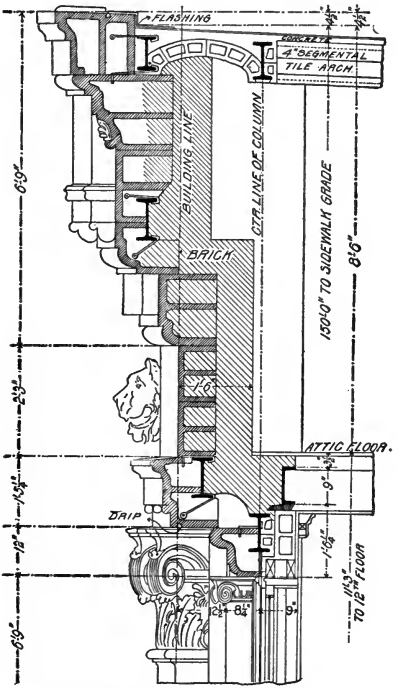

FIG. 84.—Spandrel Section, Roof and Cornice. Fort Dearborn Building.

Anchors, Ties, etc.—The ornamental terra-cotta work, however, can seldom be supported directly by the spandrel-beams, and a system of anchors must be resorted to, to properly tie the individual blocks either to the brick backing or to the metal-work itself. These anchors are usually made of ¼ in. square or round iron rods, which are hooked into the

ribs provided in the terra-cotta blocks, and then drawn tight to the brickwork or metal-work by means of nuts and screw-ends. Such anchors are shown in Fig. 86. Hook-bolts are also largely used, as in Fig. 82, where the ends are shown bent around the spandrel-channels or I-beams. Clamps are frequently employed where the terra-cotta block lies snugly against a metal flange, as indicated in Fig. 86. The many

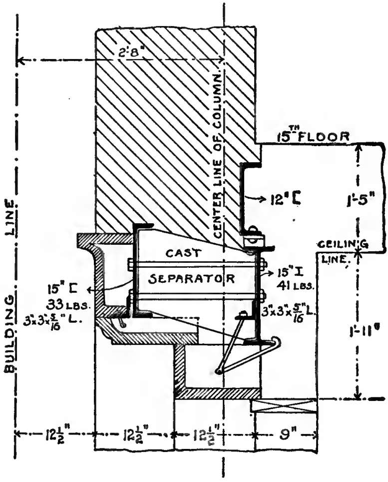

FIG. 85.—Spandrel Section. Marquette Building, Chicago.

possible methods which may be employed in securing proper anchorage cannot always be shown by drawings, and a proper execution of the work can only be secured by most careful superintendence, and study in the field. The general scheme, however, must always be indicated on the spandrel sections, as the holes necessary in the structural metal-work to receive the anchors should be included in the detail drawings of the iron- or steel-work, in order that such punching may be done at the shop.

Typical Spandrel Sections.—Fig. 85 shows a spandrel section from the Marquette Building, at the fifteenth-floor level. Heavy separators were used between the I-beam girder and the outside spandrel channel.

A rather complicated spandrel section is that indicated in Fig. 86, taken from the Marshall Field retail store building.

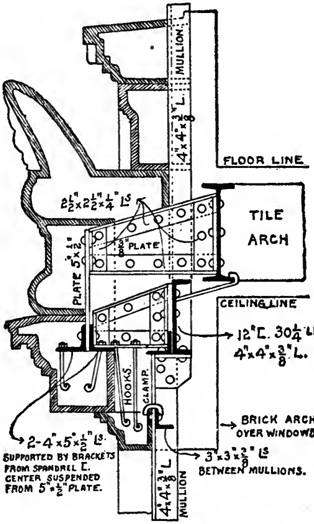

FIG. 86.—Spandrel Section. Marshall Field Building.

The spandrel-beams were here carried by the masonry piers used in the exterior walls. The section shown is taken where small ornamental balconies occur in the recessed wall between the piers. The vertical mullion-angles are plainly shown.

Fig. 87 is from the same building, taken at the level where the granite facing stops and the brick and terra-cotta work begins.

A spandrel section at the eighteenth-floor level of the

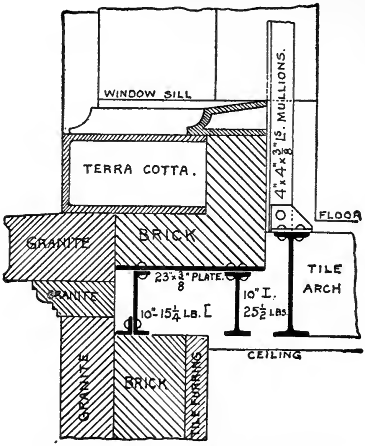

FIG. 87.—Spandrel Section. Marshall Field Building.

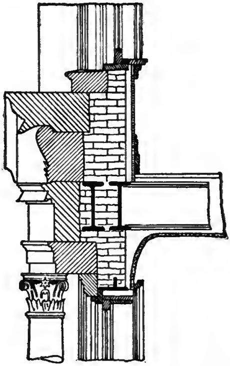

FIG. 88.—Spandrel Section, Eighteenth Floor. American Surety Co.'s Building, New York.

American Surety Co.'s Building, New York, is shown in Fig. 88. In this building, the entire fronts are constructed of granite, and the granite lintel over the window-space is shown as supporting the courses above.

Fig. 89 illustrates the construction of the cornice at the twentieth-floor level of the same building.

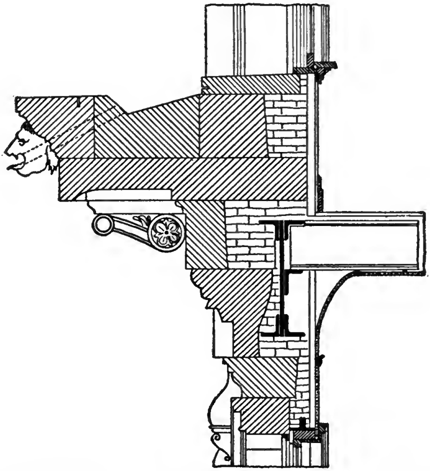

FIG. 89.—Spandrel Section, Twentieth Floor. American Surety Co.'s Building, New York.

A section through an end bay of the Gillender Building, New York, at the fourth-floor level, is given in Fig. 90. The lattice girder here shown in section is also shown in plan and elevation in previous Fig. 60 (framing plan).

Fig. 91 shows the overhanging cornice at the fifteenth-floor level of the Spreckels Building, San Francisco, Cal. The hook-bolts and clamps used to secure the marble cornice-stones are plainly indicated.

A spandrel employed at the sixteenth-floor level of the

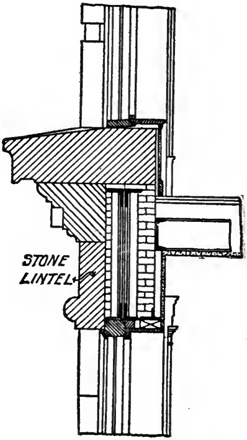

FIG. 90.—Spandrel Section, Fourth Floor. Gillender Building, New York.

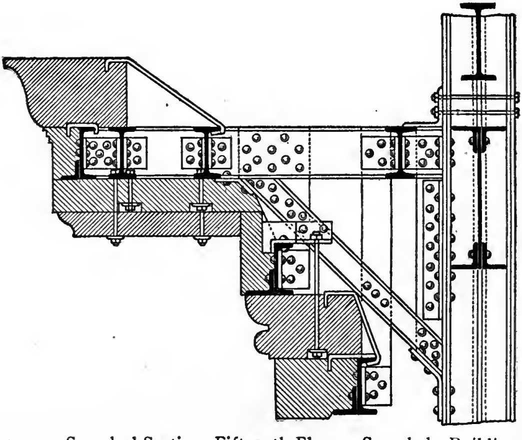

FIG. 91.—Spandrel Section, Fifteenth Floor. Spreckels Building, San Francisco.

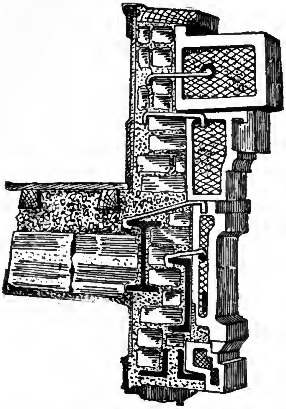

FIG. 92.—Spandrel Section, Sixteenth Floor. Broadway Chambers, New York.

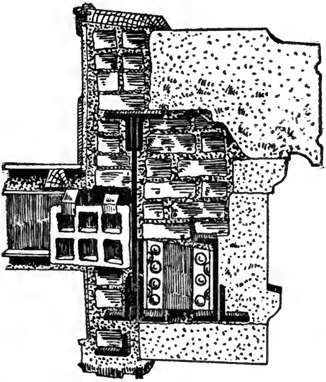

FIG. 93.—Spandrel Section, Fourth Floor. Broadway Chambers, New York.

Broadway Chambers, New York, is shown in Fig. 92, while Fig. 93 is from the same building at the fourth-floor level, showing the termination of the granite used in the lower three stories.

Court Walls.—The spandrel sections of the court walls differ in no way, as far as general principles are concerned, from those of the exterior walls. They are generally simpler, however, due to the plainer character of the wall, and to their usual decrease in thickness as compared to the exterior walls. A glazed brick is commonly employed to reflect all possible light, while the sill-courses, etc., are of terra-cotta as before.

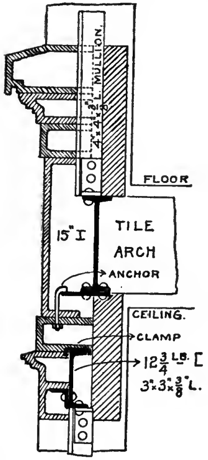

Fig. 94.—Spandrel Section, Court Walls. Marshall Field Building.

A section of the court wall in the Marshall Field Building is given in Fig. 94.

A simple court-wall spandrel section is shown in Fig. 95.

Some extremely simple and well-designed spandrel sections for court walls are shown in Figs. 96, 97, and 98, these being taken from the Cable Building, Chicago, 1899.* They represent about as simple wall construction as can be devised, and in court and alley walls a single beam and a Z-bar or possibly angle-iron, will usually provide sufficient support for the plain character of spandrels required. These sections illustrate very commendable methods of fireproofing lintels or spandrel-beams, and if similar details are employed for all spandrel sections, the severest of test conditions by fire will undoubtedly be met successfully.

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* See author's "Fireproofing of Steel Buildings."

Bay Windows.—With the introduction of steel construction and veneer methods, came the demand and possibility of con-

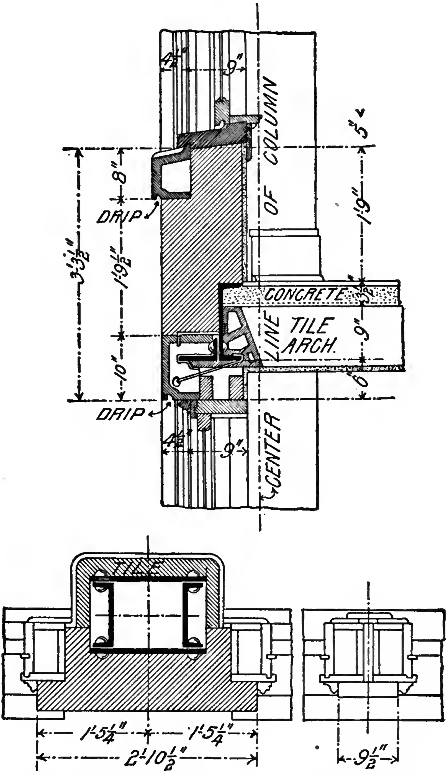

FIG. 95.—Spandrel Section. Typical Court Wall.

structing the bay window, a feature which has become more or less prominent in modern office building and hotel design.

As in the ordinary spandrel section, the material for each story must be carried in such a manner as to make it independ-

ent of the other stories. This is accomplished by means of brackets at each floor-level, and in order that the bracket loads may not become too heavy the bay-window walls must be

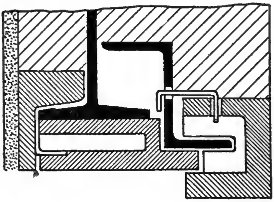

FIG. 96.—Lintel Section, Court Windows. Cable Building, Chicago.

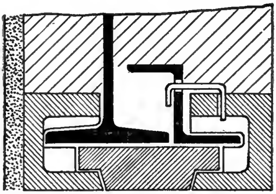

FIG. 97.—Lintel Section, Court Opening. Cable Building.

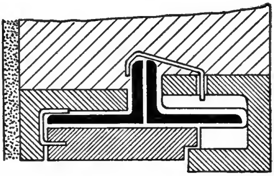

FIG. 98.—Lintel Section Alley Windows. Cable Building.

constructed as light as possible. No yielding or deflection is permissible in these brackets, and if the supporting member is a floor-beam or floor-girder, as in Fig. 99, taken through a bay window of the Masonic Temple, the girder should be rigidly connected to the floor system, to prevent any twisting

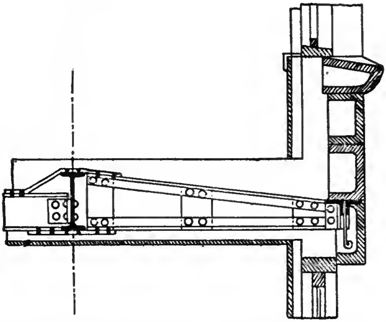

FIG. 99.—Spandrel Section through Bay Window Masonic Temple, Chicago.

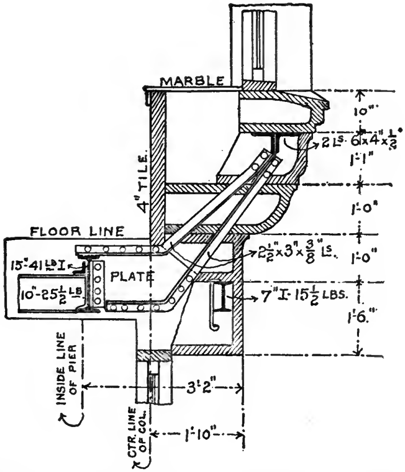

FIG. 100.—Spandrel Section at bottom of Bay Window. Masonic Temple.

tendency due to the weight of the bay. This is accomplished, as in the above-mentioned figure, by means of the top and bottom tie-plates shown.

Fig. 100 shows a section at the bottom of a bay window in the Masonic Temple.

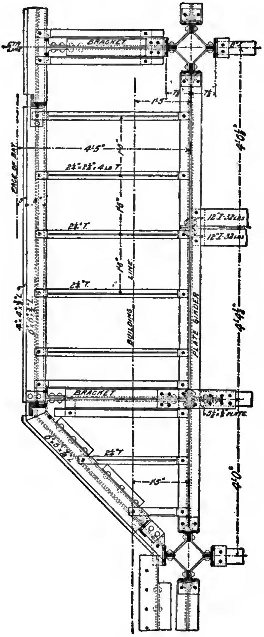

FIG. 101.—Half Plan of Framing for Bay Window. Reliance Building.

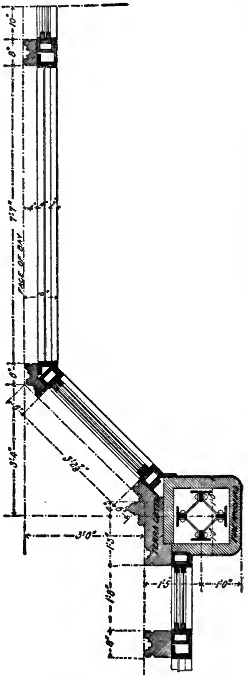

FIG. 102. — Half Plan through Bay Window Walls. Reliance Building.

Fig. 101 shows a half plan of the metal framing for the State Street bay window in the Reliance Building.

The terra-cotta mullions of the bay and the pier are shown in plan in Fig. 102.

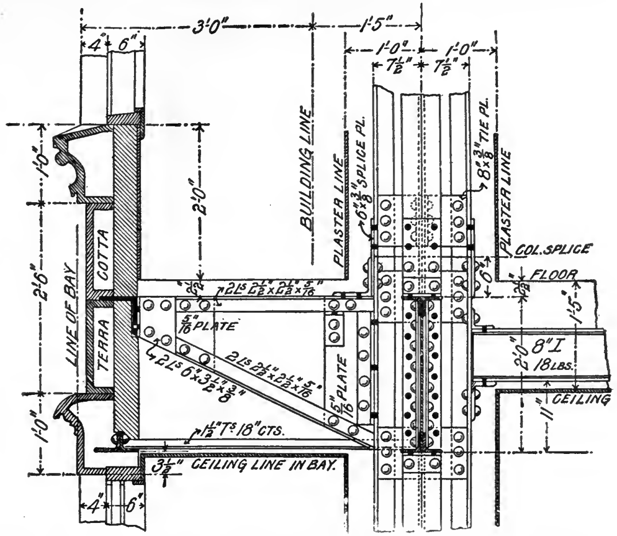

FIG. 103.—Spandrel Section through Centre of Bay Window. Reliance Building.

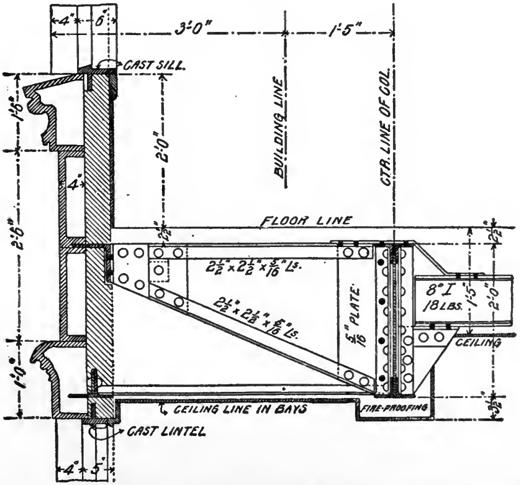

FIG. 104.—Spandrel Section at side of Bay Window. Reliance Building.

The column bracket in the bay is given in Fig. 103, while Fig. 104 is a section at the side bracket.

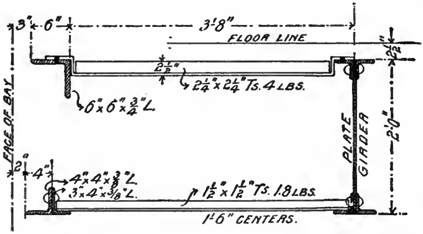

FIG. 105.—Floor and Ceiling Supports in Bay Window. Reliance Building.

The method of supporting the floors and ceilings in the bays is shown in Fig. 105.

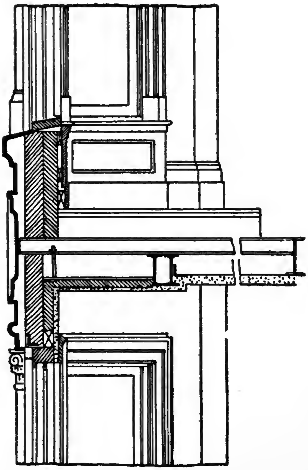

FIG. 106.—Section through Bay Windows, Fifth to Eleventh Floors. Gillender Building.

A section through one of the bay windows in the Gillender Building, fifth to eleventh floors inclusive, is shown in Fig.

106. The plan is shown in Fig. 107, while the steel framing detail is illustrated in Fig. 108. The latter should be com-

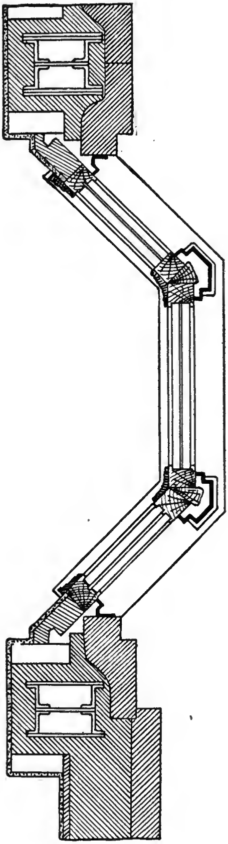

FIG. 107.—Plan of Bay Windows, Fifth to Eleventh Floors. Gillender Building.

pared with the general floor-framing plan shown in previous Fig. 60.

Calculation of Spandrel Members.—In veneer construction the masonry or piers around the columns is almost always carried on brackets, shelf-angles, or plates attached to the steel columns at each floor-level. This leaves the spandrel members to carry the curtain or spandrel walls, which lie between the piers and extend from the top or head of one

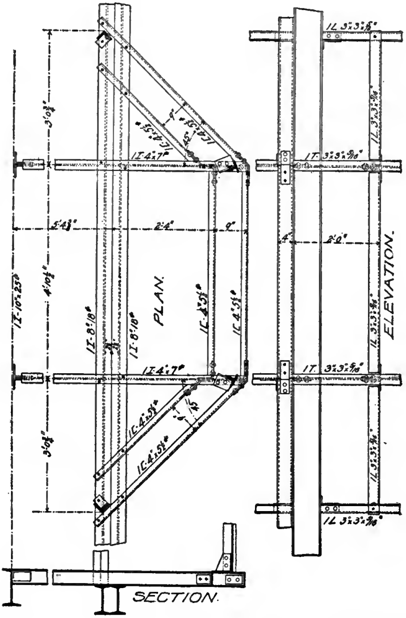

FIG. 108.—Plan of Bay window Framing. Gillender Building.

window to the head of the next higher window. The spandrel framing members are, therefore, to be calculated for the uniformly distributed wall- and window-loads which they carry, in precisely the same manner as explained for floor-joists. A table of weights of material, useful in such calculations, is

given at the end of this chapter. If several spandrel members are used, at somewhat different levels, and for distinctly different conditions or magnitude of loading, each piece should be calculated independently, as, for instance, in Fig. 84. Or, if several like members are to be used side by side, they may be considered as subject to one uniform loading, each piece to carry its proportional share of the total.

If the floor arch is also to be carried by the spandrel member, in addition to the spandrel load, as in Fig. 94, the total load must be figured—or a uniform load per foot consisting of the spandrel weight plus the floor-load due to one-half the floor arch adjacent to the wall.

Spandrel members with brackets, as in Fig. 86, must be calculated for concentrated loads, while bay-window brackets, etc., must be figured as cantilevers, with especial attention given to the flange connections with the supporting floor-beam or girder.

If the window areas are narrow, and the piers wide, with the latter partially supported by the spandrel beams, such pier-loads may be considered as concentrated at the centre lines of the portions resting on the spandrel members, and provided for accordingly in addition to the uniform load of the window width.

Lintels.—For openings in interior or exterior walls where lintel beams or members are supported directly by the walls, without any connections to columns, the load generally assumed to be carried, in masonry of usual bond, may be represented by a triangle whose base equals the clear span, and whose height equals one-third of the span, see Fig. 109. If openings occur in the wall, as shown in the figure, the load is usually assumed to be the wall area included within the outside heavy dotted lines shown.

Two or more beams bolted together with cast-iron separators, and resting on cast-iron or steel bearing-plates at the

ends, are usually employed to insure lateral rigidity and better bearing for the wall to be carried. The following table * gives

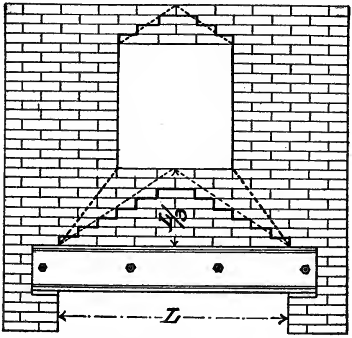

FIG. 109.—Lintels in Masonry Walls.

suitable beams for openings in properly bonded solid brick walls, with deflections less than $\frac{1}{360}$ of the span up to 10 ft., and $\frac{1}{500}$ of the span if from 15 to 20 ft.:

Thickness of Wall in inches.	Spans in Feet.					
	8 or 9 ft.	10 or 11 ft.	12 or 13 ft.	14 or 15 ft.	16 or 17 ft.	18 or 20 ft.
9	2-4″ $7\frac{1}{2}$ lb.	2-5″ $9\frac{3}{4}$ lb.	2-7″ 15 lb	2-8″ 18 lb.	2-9″ 21 lb.	2-12″ $31\frac{1}{2}$ lb
13	2-4″ $7\frac{1}{2}$ lb.	2-6″ $12\frac{1}{4}$ lb.	2-7″ 15 lb.	2-8″ 18 lb.	2-9″ 21 lb.	2-12″ $31\frac{1}{2}$ lb.
18	2-5″ $9\frac{3}{4}$ lb.	2-7″ 15 lb.	2-8″ 18 lb.	2-9″ 21 lb.	2-10″ 25 lb.	2-12″ $31\frac{1}{2}$ lb.
22	2-5″ $9\frac{3}{4}$ lb.	2-7″ 15 lb.	2-8″ 18 lb.	2-9″ 21 lb.	2-10″ 25 lb.	2-12″ $31\frac{1}{2}$ lb.

Cast-iron lintels may be computed as follows: assume a $\perp$ section, the horizontal member of which is 12 ins. wide by 1 in. thick, and the vertical web of which is 7 ins. high by 2 ins. thick. The lintel is therefore 12 ins. broad, and 8 ins. high. Assume the clear span as 8 ft. 0 ins.

The neutral axis may then be computed from the base line of the lintel; or

$$y_1 = \frac{(2 \times 7)4 + (12 \times 1)\frac{1}{2}}{14 + 12} = 2.38 \text{ ins.}$$

* See "Steel in Construction," issued by A. & P. Roberts Co.

The neutral axis is, therefore, 2.38 ins. up from the base line, and 5.62 ins. down from top of web. I then equals

$$\frac{2 \times \overline{5.62}^3 + 12 \times \overline{2.38}^3 - 10 \times \overline{1.38}^3}{3} = 163.$$

$M = \frac{Wl}{8}$ for a uniformly distributed load, and, as $l = 96$ ins., M therefore equals $12\,W$ inch-pounds.

But $M = \frac{fI}{y_1}$, and f for the upper fibres in compression may be taken at $\frac{90,000}{6} = 15,000$ lbs. Hence,

$$M = 12W = \frac{15,000 \times 163}{5.62} = 433,000 \text{ lbs.}$$

$$W = 36,000 \text{ lbs.}$$

For the lower or tension fibres, $f = \frac{20,000}{6} = 3,334$ lbs. Hence,

$$M = 12W = \frac{3,334 \times 163}{2.38} = 228,000 \text{ lbs.}$$

$$W = 19,000 \text{ lbs.}$$

Hence, the safe distributed load for a factor of safety of 6 in tension, should not exceed 19,000 lbs.

TABLE OF WEIGHTS USED IN THE CALCULATION OF SPANDREL LOADS, PIER LOADS, ETC.

Brick masonry, common brick	112	lbs. per cubic foot.
" " pressed brick	140	" " " "
" " hollow brick	90	" " " "
Concrete, cinder	84	" " " "
" stone	150	" " " "
Masonry, bluestone	160	" " " "
" granite	170	" " " "
" limestone	160	" " " "
" marble	160	" " " "
" sandstone	144	" " " "
" slate	160	" " " "
Terra-cotta, brick backing	112	" " " "
Glass, sash, etc.	5	lbs. per square foot.
Plaster, on terra-cotta arches	5	" " " "
" on lath	7	" " " "
Slate, on roofs, etc., laid	6	" " " "
Snow, fresh-fallen	7	" " " "
" wet and packed	15 to 50	" " " "
Skylights	50	" " " "

WEIGHTS PER SUPERFICIAL FOOT FOR BRICK WALLS OF DIFFERENT THICKNESSES.

(On a basis of 112 lbs. per cubic foot.)

9-inch wall	84	lbs. per superficial foot.
13-inch "	121	" " " "
17-inch "	168	" " " "
21-inch "	205	" " " "
25-inch "	243	" " " "
29-inch "	289	" " " "
33-inch "	326	" " " "
38-inch "	373	" " " "
42-inch "	410	" " " "
46-inch "	448	" " " "
50-inch "	486	" " " "
54-inch "	532	" " " "

CHAPTER VII.

COLUMNS.

THE subject of the interior columns forms one of the most important steps in the modern problem of design, and greater variations are probably to be found here than in any other of the vital features in iron or steel construction. The many forms of columns now in the building market, each having its own good points, and the many types of connections between the columns themselves and with the floor system, permit of a choice from a dozen or more types, with the details varying widely in each case, to suit the shape chosen. We shall endeavor to investigate the more prominent forms, and point out the advantages and disadvantages of each one. The most satisfactory for general or specific cases may then be selected, as combining the features desired.

Cast-iron Columns.—A discussion as to the relative values of cast *versus* steel columns should hardly seem necessary at the present time, but the repeated use of the cast-iron column in ten- to sixteen-storied buildings, and even higher (as exemplified by their use in the Manhattan Life Insurance Building of seventeen stories), shows that the questionable economy of cast columns does still, in the opinion of some architects, compensate for the dangers incident to their use. The best practice has declared so uniformly, during the last few years, in favor of the steel columns that the employment of cast metal is now pretty generally confined to buildings of very moderate height or to special cases where advantages are to be gained,

as in the use of a number of ornamental cast columns. The great uncertainty as to the uniformity of cast metal led to the use of a very low unit-stress, while in the case of steel the unit-stresses can be assumed on a very definite reliance on the trustworthiness of the metal. Among more progressive designers the use of cast-iron in large buildings has become a thing of the past, and would no more be seriously considered than would the use of cast-iron compression-members in bridges.

Considering the cast sections in more general use as columns, the circular, square, and H-shaped, and their individual connections (see Fig. 110), it will be seen that these splices cannot result in as rigid a framework as the riveted joints in steel-work. The columns in the modern design must be capable of affording stiff connections so as to withstand both the direct dead- and live-loads transferred from the floor system, as well as sufficient connections for the wind-bracing. These cannot be secured well by means of bolts passing through the horizontal flanges of cast columns, even if the workmanship be considered accurate. The workmanship, however, can seldom, if ever, be relied upon as perfect; the bolts never completely fill their holes, and "shims" are constantly employed to plumb the columns. These constitute elements of weakness which may easily allow considerable distortion. The girder connections to the columns, resting on cast brackets and bolted through the flanges, are bad in the extreme, especially for cases of eccentric loading and the irregular placing of beams.

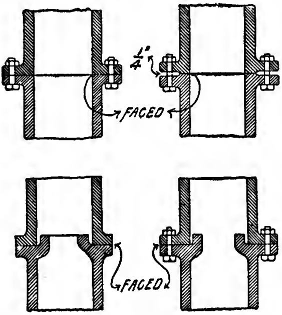

FIG. 110.—Details of Splices for Cast-iron Columns.

To offset these dangers of weak design it is true that cast columns are cheaper per pound and perhaps easier of erection than the steel—considerations that naturally have much weight with the owner of the building. But considering the risks that are run, as in the building at 14 Maiden Lane, New York, which was blown eleven inches out of plumb through the inability of the cast columns to resist the wind pressure, it is hard to understand why architects will persist in the use of such methods, even if requested by the owner. Cast-iron, in spite of its apparent stiffness, has a much lower coefficient of elasticity than steel, breaking suddenly when it breaks, while steel suffers distortion.

Steel is now being rolled at such a low price that, considering the extra weight necessary in cast-iron, on account of its unreliability, the saving in cost by the use of the latter will be found to be small indeed, even disregarding the dangers assumed by its use.

The formula ordinarily used in proportioning cast-iron columns, and commonly known as Gordon's or Tredgold's formula, is

$$p = \frac{80,000}{1 + \frac{1}{400}\frac{l^2}{d^2}}.$$

The only basis for this formula, or for the same form with different coefficients as used by various writers, consists of a series of tests made by Hodgkinson in about 1840 on nine so-called "long" pillars, and thirteen "short" pillars. The long specimens were 7 ft. 6 ins. in length, with external diameters ranging from $1\frac{3}{4}$ to $2\frac{1}{4}$ ins., while the short pillars were not over 2 ft. 6 ins. long, with external diameters of 1 to $1\frac{1}{4}$ ins., and a thickness of metal in no case exceeding $\frac{1}{4}$ in. Considering the nature of cast-iron, and the methods of manufacture employed in making large cast columns, it is evident that any such experiments as the above are in no way suitable

to form the basis for any formulæ to be used in proportioning members of such size as ordinarily enter into building construction. For this reason, the use of cast-iron members in bridge construction has not been countenanced by civil engineers for more than twenty years past, yet Gordon's formula has continued in use for building work, and, until 1899, the formula given above has been practically required by the New York Building Law.

During the past few years, however, additional tests have been made on full-sized sections—including the tests of Prof. Lanza at the Watertown Arsenal, and the later and more important tests made at Phœnixville, Pa., by the New York Building Department in 1896 and 1897; and although these experiments do not cover any great range of sectional forms or of the ratio of length to diameter, still the results are sufficient to show the complete unreliability of the formulæ commonly employed.

According to the Phœnixville tests, 15-in. columns which, by Gordon's formula, should possess a breaking strength of 57,143 lbs. per sq. in., failed under stresses varying from 24,900 lbs. to about 40,400 lbs. per sq. in., while 6-in. and 8-in. columns, with a calculated strength of 40,000 lbs. per sq. in., showed a breaking strength of from 22,000 lbs. to 31,900 lbs. per sq. in. only.*

From the foregoing tests, Prof. Wm. H. Burr has deduced the straight-line formula

$$p = 30,500 - 160\frac{l}{d},$$

where p equals the ultimate resistance per square inch. This gives about a mean of the tests as plotted, "and represents as near as any that can be found, a reasonable law of variation of ultimate resistance with the ratio of length over diameter."

* See *Engineering News*, Jan. 20, 1898.

The plotted values of the formula

$$p = 52,500 - 563\frac{l}{d}.$$

determined by actual tests made on mild steel angles by Mr. James Christie of the Pencoyd Iron Works, "show that the ultimate resistances per square inch of mild steel columns are from 40 to 50 per cent. greater than the corresponding quantities for cast-iron, the same ratio of length over diameter being taken in each comparison." *

Prof. Burr gives as his conclusion:

"When the erratic and unreliable character of cast-iron is considered, it is no material exaggeration to state that these tests show that the working resistance per square inch may probably be taken twice as great for mild steel columns as for cast-iron; indeed, this may be put as a reasonably accurate statement.

"The series of tests of cast-iron columns represented in the plate largely destroys confidence in the cast-iron column design of the past. The results of the tests constitute a revelation of a not very assuring character in reference to cast-iron columns now standing, which may be loaded approximately up to specification amounts. They further show that, if cast-iron columns are designed with anything like a reasonable and real margin of safety, the amount of metal required dissipates any supposed economy over columns of mild steel. As a matter of fact, these results conclusively confirm what civil engineers have long known, that the use of cast-iron columns cannot be justified on any reasonable ground whatever."

Steel Columns. — The more prominent forms of steel columns as used in American building practice include channels connected by plates or lattice, plates and angles in various

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* See Prof. Wm. H. Burr in *School of Mines Quarterly*, April, 1898; also in *Engineering News*, June 30, 1898.

forms, and Z-bar columns. Besides these types, and the considerable number of variations found in each, special forms such as the Keystone Octagonal, Phœnix, Larimer, and Gray columns have been used to more or less extent, but these patented or restricted forms have not been employed as extensively as those columns which are made of shapes in common use, without any restrictions as to patent rights or availability of material.

Channel Columns.—Ordinary forms of channel columns are shown in Fig. 111. For light members, as in upper

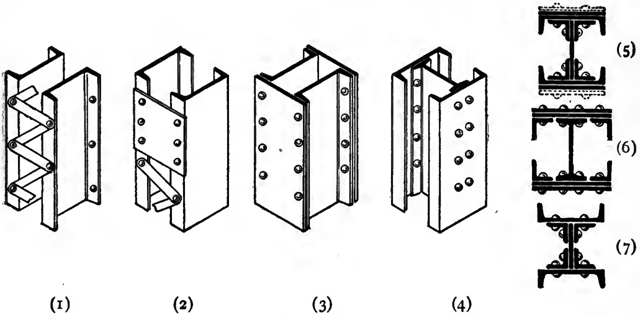


FIG. 111.—Typical Forms of Steel Channel Columns.

stories, the channels are often placed back to back or flange to flange, and connected by means of tie-plates and lattice bars. The former method of placing the channels back to back is somewhat easier as regards the riveting. The third form shown, with cover plates either single or double, is one of the most common column sections employed. The fourth form shows a combination of two channels and an I-beam. A variation of this section is sometimes made by substituting a plate and four angles in place of the I-beam, or one or more plates and two angles for the channel sections. These forms were used in the Harrison Building, Phila., and are shown in the sections 5, 6, and 7.

Plate and Angle Columns.—Typical forms of plate and angle columns are shown in Fig. 112. The simplest combina-

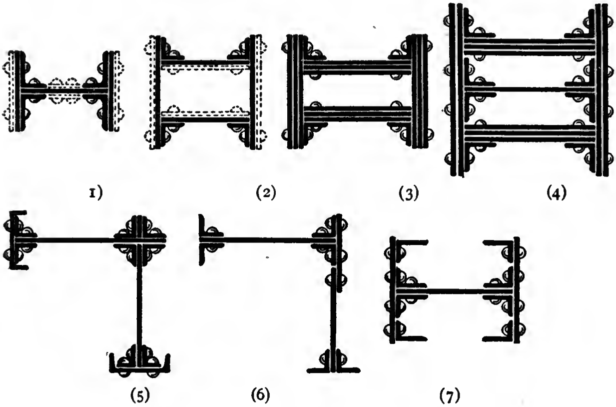
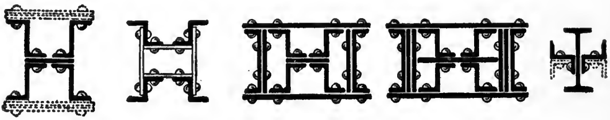


FIG. 112.—Typical Forms of Plate and Angle Columns.

tion is that made in the form of a beam. One or more webs may be used, or fillers between the angles as shown by the dotted lines, but any additional material is placed to better advantage if used in the form of cover-plates, riveted to the outer legs of the angles. The I section of plates and angles is extensively used in cases where the loads are sufficiently light to permit of its use. This form of column was used in the Manhattan Life Building, New York City. The box form of plates and angles, shown as the second type in the illustration, is one of the most ordinary as well as commendable forms in common use. This section may be readily strengthened by using additional web-plates, cover-plates, or filler-plates, as illustrated by the dotted lines, or by section 3. Columns of this form have been used in a great many notable high buildings; as, for example, the St. Paul, the American Surety, and the Park Row Buildings in New York City, and the Masonic

Temple in Chicago. Section 4 illustrates a particularly heavy section employed in the Manhattan Life Building. Special corner wall-columns used in the Dun Building, New York, and in the Worthington Building, Boston, are shown in forms 5 and 6 of Fig. 112, while form 7 shows a variation of the beam and channel column, as used in the Harrison Building, Phila.

Z-bar Columns.—Z-bar columns and variations are shown in Fig. 113. The ordinary section is as in form 1, this being



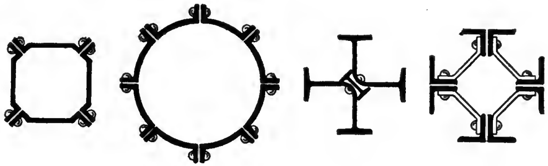
(1) (2) (3) (4) (5)

FIG. 113.—Typical Forms of Z-bar Columns.

made in the standard sizes of 6-in., 8-in., 10-in., and 12-in. columns, by using 3-in., 4-in., 5-in., and 6-in. Zees respectively. When the load can be safely carried without the aid of cover-plates, and if the size of the column does not become too large for its relative position in the building, it is more economical to use the simple section, but when additional area is required, one or more cover-plates may be added as shown by the dotted lines. Form 2, known as the "standard dimension" Z-bar column, was designed to allow of the outside dimensions of such columns being kept standard for all stories, irrespective of the size or thickness of Z's required, but on account of the tie-plates required in either one or both directions increasing the shop costs, and decreasing the efficiency of the column under eccentric loading, the form has never come into extensive use. Sections 3 and 4 show heavy columns combining Zees with plates and channels. These forms were used in the Manhattan Life Building. Section 5 shows a combination of

two Z-bars with one I-beam, as used in the Dubuque, Iowa, Bank Building. The ordinary sections were made of 10-in. I-beams and 5-in. Z-bars, while in the heavier sections the Zees were reinforced by angles, as shown in the dotted lines.

Special Columns.—Fig. 114 illustrates what may be



(1) (2) (3) (4)

FIG. 114.—Special Forms of Steel Columns.

termed special forms of steel columns, inasmuch as these sections are either controlled by patents, or else their manufacture is restricted to certain mills which roll the special shapes required. Form 1 shows the Keystone Octagonal column, which is now rarely, if ever, seen in building practice. The Phœnix column, form 2, will be discussed in a later portion of this chapter. This form has many commendable points, but the special shapes of material required restrict its manufacture to certain mills. The Larimer column, shown in form 3, is controlled by Jones & Laughlins, L'd., while the Gray column, form 4, is still controlled by patents.

The foregoing examples will serve to show the great number of forms offered the designer, from which a selection must be made. Nearly all of the sections illustrated, save the Keystone Octagonal column, are to be found in prominent examples of building construction, while various other special forms or combinations have been proposed or actually employed. These latter may, however, be classed as curiosities, or designs dependent upon very special conditions.

Theoretical Requirements in Column Design.—The relative advantages of these standard sections are, obviously, of considerable importance in influencing a choice; but that any particular type can be selected as the best for universal application, is manifestly impossible. In actual practice the treatment of these different shapes will be found to vary greatly with the designer—not only in the relative value of the sections, but in the treatment of any one section. In the first place, column formulæ differ greatly, not in fundamental principles perhaps, but in the treatment, being often empirical, and containing factors deduced from some special case. These formulæ also generally assume ideal loading, which will seldom occur in the modern building, and practically no full-sized tests have ever been made on the effects of eccentric loading. Full-sized tests, on columns of concentric loads even, have been far too limited to show the relative values of the most ordinary column sections.

Prof. Burr, in his "Strength and Resistance of Materials," states that "The general principles which govern the resistance of built columns may be summed up as follows:

"The material should be disposed as far as possible from the neutral axis of the cross-section, thereby increasing R;

"There should be no initial internal stress;

"The individual portions of the column should be mutually supporting;

"The individual portions of the column should be so firmly secured to each other that no relative motion can take place, in order that the column may fail as a whole, thus maintaining the original value of R."

The experiments given by Prof. Burr would indicate that a closed column is stronger than an open one, due to the fact that the edges of the segments are mutually supporting when held in contact by complete closure. From a theoretical standpoint, therefore, the Phœnix column is undoubtedly the most

favorable form for compression, as it forms a closed, and thus mutually supporting section; and because the capacity of columns of equal areas varies as the metal is removed from the neutral axis. It must also be remembered that any form of column having a maximum and minimum radius of gyration is not economical for use under a single concentric load, as the calculations must be based on the minimum radius of gyration. The metal represented by the excess of the maximum radius of gyration is of necessity disregarded, and part of the section is thus lost or wasted, when we consider the ideal efficiency of the column. But practice does not always support theory, and many other questions besides mere form arise in connection with the judicious choice of a section. Indeed, it will be seen that several practical considerations in the use of columns in buildings call for a form very different from the ideal circular section; such points as the transfer of loads to the centre of the section, the maximum efficiency under eccentric loading, and the requirements for pipe-space around or included in the column form, all tend seriously to restrict the use of closed or circular sections.

Ordinary Column Formula.—To determine the relative importance of these practical considerations to the theoretical requirements of column design, consider the formula

$$p = \frac{f}{1 + a\frac{l^2}{r^2} + \frac{x_0 y_1}{r^2}},$$

where p = ultimate strength in pounds per square inch;
f = elastic limit of the material in compression;
a = constant, varying according to end bearings;
l = length of column in inches;
r = radius of gyration of cross-section in inches;
x_0 = distance of application of eccentric load from centre of gravity of the column section, in inches;
y_1 = distance of extreme fibres from centre of gravity of column section, in inches.

This is the form of Gordon's or Rankine's formula for columns, including the effect of eccentric loading, besides the expressions for the strains in the column due to the uniformly distributed load, and those due to the flexure of the column. The term for eccentric loading does not occur in the ordinary form of Gordon's formula, but in building construction this term must not be neglected in considering the relative importance of the strains to which the great majority of building columns are subjected. The girder loads are necessarily applied to the sides of the columns, and unless these loads are equal, and on opposite sides of the columns, the eccentricity of the resultant load tends to increase the strains on the side where the greater load occurs.

Considering now the three terms in the denominator of the previous value for p, the first, namely 1, or $\frac{f}{1}$, represents the strain due to the uniformly distributed or concentric load. This, of course, is the principal strain to which the column is subjected, and in short columns, with perfectly concentric loads, would represent the only load or condition to be used in proportioning the number against crushing.

The second term in the denominator, $a\frac{l^2}{p^2}$, representing the strain due to the flexure or bending of the column, is usually so small that it really makes this term of the least importance in the above equation, due to the ordinarily short lengths of columns in buildings, and to the fact that the bases or ends are broad and flat bearing. Prof. J. B. Johnson shows* from examples selected from actual building practice in a sixteen-story building, that the value of this term varies from 0.022 in the basement columns, to 0.220 in the smallest columns of reduced section at the top of the building.

* See "Modern Framed Structures," page 451.

The third term of the denominator, namely $\frac{x_0 y_1}{r^2}$, or the expression for the strains due to eccentric loading, is shown by Prof. Johnson to be more important than considerations as to flexure. He gives an ordinary value for this term of 0.07 or more in basement columns, taken from the same building example previously quoted, while in the columns for the upper floors the value is shown to be 0.6 or 0.7, which, in these small-section columns, "occasionally doubles the section."

These figures "show that the important effects of eccentricity of loading increase rapidly as the section of the column decreases, and that the importance of this element in columns thus eccentrically loaded is three or more times as great as that of the element dependent upon the flexure of the column. These effects are entirely independent of the character of the column, varying of course in values with different kinds of columns, but always true when the loading is as irregular and eccentric as the architecture of modern high buildings necessitates."

In columns of one-story lengths, therefore, where the length is usually under 90 radii, considerations as to flexure may generally be disregarded, and the differences in the ideal strengths of the various sections tend to disappear. If the columns are well made, and subject to concentric loads only, almost any of the ordinary column sections will give satisfactory results if used with ordinary unit stresses. And by far the larger number of columns used in modern building construction is under 90 radii, as they are used in single-story lengths of from 10 to 14 ft. The determining factors in a selection are, therefore, such practical considerations as effect columns of these lengths; so that the ideal disposition of the metal must be considered in connection with other very important requirements.

Practical Requirements in Column Design.—The following elements of design should be carefully considered:

1. Cost, availability.

2. Shopwork, and workmanship of column.

3. Ability to transfer loads to centre of column—eccentric loading.

4. Convenient connections. Splices.

5. Relation of size of section to small columns.

6. Fireproofing capabilities of the section.

Points 1 and 2 are of the greatest importance to the owner and builder, and often govern the selection of the column. Points 3, 4, and 5 are for the engineer's consideration, while point 6 is of chief interest to the architect and decorator.

Cost, Availability. — The question of the cost of the material as it comes from the mill is a purely commercial one, depending upon the market price per pound of the section used.

The prices of plain beams, channels, Zees, plates, and angles, vary from time to time, as fixed by agreement among the steel producers, and while Zees may sometimes be more expensive than beams and channels, at other periods it will be found that there is no great difference, if any, between the more ordinary marketable shapes. The cost of the raw material, however, will practically never determine the relative costs between various column forms, as the expense of manufacture, the weight of the columns, and the question of simple *vs.* complex details and the duplication of members, will all influence the ultimate cost to much greater extents than the simple cost of the plain material. All of the special columns, such as the Phœnix, Keystone Octagonal, Larimer, and Gray forms, have the great disadvantage of being rolled or manufactured by certain mills only, and the quickest possible delivery of material is a very essential point. The demands for structural steel at good seasons of trade in this country are

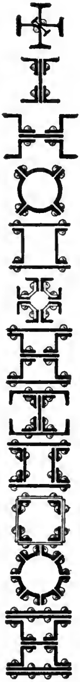
so great that it is often next to impossible to secure such prompt delivery of material as is required for the completion of a large building within the contract time. The contracts that have been executed in American cities during the last three or four years have undoubtedly shown the most wonderful construction in points of excellence and time that the world has ever seen; and it is said of a large building in New York City that the masonry for the twelfth story was laid before the mortar at the first-floor level was dry.

The steelwork for a building of any considerable size is almost invariably rolled to order, and the best arrangements as to time of deliveries can be made when the plans call for such shapes as are manufactured by several competing mills. The conditions of orders or contracts in hand may preclude the possibility of quick deliveries by certain mills or shops, while if the material be of common marketable forms, the contract may be placed to advantage with other parties better able to name the required time agreements.

The Phœnix shape, although the patent has long expired, is rolled by but one mill in this country. The Keystone column was also controlled by one particular mill, but this section is now seldom, if ever, used. The Larimer column is controlled and manufactured by a single mill, while the Gray column is made of angles and is consequently easy to obtain as to material, and the shop labor may easily be executed by any first-class plant, but the privilege of use must be secured at an additional cost.

Advantages as to availability are therefore possessed by the columns which can be most readily bought and manufactured, and there is consequently little difference between any of the forms shown in Figs. 111, 112, and 113, provided the sizes and weights of material are limited to the more ordinary varieties.

Shopwork and Workmanship.—With the present uniform low price per pound of most of the steel sections, the items of shopwork and workmanship become of far greater importance



Larimer column, 1 row of rivets.

Plate and angles, 2 rows.

Z-bar column, without covers, 2 rows.

4-section Phœnix column, 4 rows.

Channel column, with plates or lattice, 4 rows.

Gray column, 4 rows.

Z-bar column, with single covers, 6 rows.

Channels, web-plate, and angles, 6 rows.

Box column of plates and angles, 8 rows.

Latticed angles, 8 rows.

8-section Phœnix column, 8 rows.

Z-bar column with double covers, 10 rows.

FIG. 115.—Column Forms, Showing Required Punching Operations.

in the cost of the completed column than the cost of the section at the mill—assuming the sectional area, and hence the weight per foot, to be the same. Lattice bars, fillers, gussets, etc., add just so much more weight, without increasing the

section, and must therefore be considered from an economical standpoint. The methods of riveting the sections together in the various forms must also be taken into account.

The number of rows of rivets required, and the consequent punching operations, are shown in Fig. 115.

The Larimer column, manufactured and controlled by Jones & Laughlins, and first used in Chicago in the Newberry Library Building, consists of two I-beam sections bent down along the middle of the web, the two beams being riveted together with a small I-beam filler between. The rivets are spaced 3-in. centres for about 18 ins. from each end of the column, and then 5-in. centres.

Where necessary to strengthen the column, this filler is made of two channel-sections, back to back, extending out on either side as far as necessary. Small angles are riveted to the faces of the I-beams, and a plate is riveted across the top, on which the girders and column rest (Fig. 116). Where only two girders occur, the remaining faces are used to rivet the upper column to the plate. Another method has been used instead of the small angles, in the shape of a square or octag-

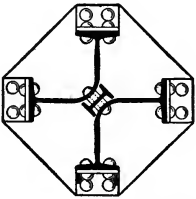


Fig. 116.—Detail of Larimer Column.

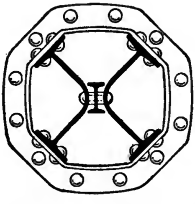


Fig. 117.—Detail of Larimer Column.

onal sheet which is cut from the centre out, part way to the edge, and the lips so formed are bent down in a press, thus making a solid and continuous angle. Still another detail has been made by pressing out in a hydraulic machine a circular

sheet to conform in the lower part to the shape of the outside of the flanges of the column (Fig. 117). In this way not only the upper flange, but the vertical flange also, is made continuous around the top of the column. Also the thickness of the horizontal flange is retained uniform, the thickness of the vertical flange being somewhat tapered.

This column possesses one great disadvantage in the smaller-sized columns. This lies in the difficulty of driving the rivets that connect the bracket angles with the I-beam flanges. In a 6-in. column, where 5-in. I-beams are used, or in smaller columns, it is often very difficult on account of interference to drive the rivets through the holes, unless the rivets are driven in a slanting direction. This often results in weak connections.

The Larimer column is not adapted to heavy work, as the form of the section does not permit of easy reinforcement under large loads. The splicing facilities are also bad, as horizontal cap-plates must invariably be used. The difficulty of shopwork in the bending of the I-beams is also very liable to result in poor workmanship, unless the greatest care is exercised; and riveting through the beam flanges is apt to contribute to shop difficulties and imperfections. In general, it may be said that all column sections composed of combinations of I-beams are difficult to manufacture, on account of the trouble in riveting through the bevelled flanges.

The Larimer column has had no very extensive use in high building construction, and is now seldom used in any important work.

The Phœnix column has been used in several prominent high buildings, notably in the "World" and Dun buildings, New York City, but on account of the difficulty of connections, which will be discussed under a later heading, this form has gradually lost favor. In the matter of shopwork, the Phœnix column has disadvantages as regards the special devices neces-

sary to secure adequate connections, and in the limitations as to the rolling of the special segmental forms.

The Gray column requires, as will be seen by Fig. 115, no less than sixteen punching operations for the four rows of rivets employed, besides the additional expense of the special shaped tie-plates which are necessary to connect the four individual struts, but which do not contribute to the effective area. This column has been used in a number of prominent buildings, including the Ellicott Square and Guaranty Buildings, Buffalo, the Reliance and Fisher Buildings, Chicago, and the Chamber of Commerce and Mabley Buildings in Detroit, but by many engineers, this form of column is not regarded with favor, as will be pointed out under the later consideration of eccentric loading.

The same objections as to the superfluous metal required by tie-plates which cannot be counted on as available sectional area, and the weakness under eccentric loading, are true of the standard dimension Z-bar column.

Columns made of latticed angles or channels are usually limited to very moderate loads in upper stories, or in buildings of no very considerable height. The lattice bars and tie-plates constitute excess material, besides contributing largely to the cost of manufacture. Columns made of channels and web-plates are very satisfactory where the loads do not become so great as to require more than one cover-plate. In heavier sections, a box column of plates and angles becomes more desirable.

For buildings of moderate height and loading, no more advantageous section can be employed than the Z-column. In such cases, the advantages of simplicity of shop labor, requiring but two rows of rivets, the availability of material, and the facility for obtaining good girder connections and column splices, outweigh the advantages possessed by any other column form. For higher buildings, or heavier loads,

where the required sectional area is greater than can be obtained by using Z-bar columns without cover-plates, the box column of plates and angles will be found most satisfactory. This column form possesses great advantages regarding connections, in that square surfaces are always presented. Box columns were used in the Masonic Temple, the highest building in Chicago, and in the Park Row Building, the highest structure in New York City. In the Masonic Temple, latticing was used on two sides of the columns in the upper stories.

The character of *workmanship* will vary somewhat with the different shops, as well as with the different sections used. The reputation of the shop, seconded by careful shop inspection, will largely determine the excellence of the workmanship.

Ability to Transfer Loads to Centre of Column—Eccentric Loading.—It will be seen at a glance that some of the sections under consideration are totally unfitted for the transfer of loads to the centre of the column. The conditions in designing a framework are seldom so favorable as not to require many of the columns to be loaded unsymmetrically, for even where equal-size girders meet on opposite sides of a column, one of them may carry a heavy live load while the other may be required to carry only the dead load of the floor system, though both are figured for the same proportion of total loads. In more extreme cases, wide variations often exist in the sizes of the opposite girders, as in the loads which they carry; and in exterior columns, unless designed with particular regard to concentric loading, eccentricity will almost always occur to a greater or less degree.

Views as to the importance and treatment of eccentric loads vary considerably with different designers. In many large and important buildings, eccentric loading has had little, if any consideration, and some writers who might properly be called authorities, hold that any very careful calculations for

eccentricity are unnecessary, inasmuch as a single eccentric load usually constitutes so small a percentage of the total load on the column, and further because the building laws in most large cities require such large factors of safety, both in the assumed loads and in the unit-stresses employed in proportioning the members.

Other authorities and designers lay particular emphasis on the treatment of eccentric loading, as the relatively short lengths of building columns is considered as being more than offset, in many if not most cases, by the conditions of eccentricity of loading. Calculations for eccentric loads are tedious and unsatisfactory as far as present formulæ are concerned, notwithstanding which they are still required in any work of importance or magnitude.

It was shown earlier in the examples quoted from Prof. Johnson's "Modern Framed Structures," that the term for eccentric loading in Gordon's or Rankine's formula was of much more importance in cases taken from actual practice than the term representing the flexure or bending, and dependent upon the length. Mr. Leopold Eidlitz, in a theoretical analysis of the strength of pillars, states that: *

"Engineering handbooks published by various rolling mills and others having the authority of competent engineers compute pillars planed top and bottom or pillars of continuous length held in position by girders and beams abutting upon them, and fastened to them with bolts or rivets at every story, as subject to compound flexure under a safe load.

"Deflection of pillars usually employed in building are exceedingly small under safe loads; for instance, deflections of wrought-iron pillars from 12 to 20 diameters long (under loads varying from 11,790 to 10,600 lbs.) are 0.003 to 0.022 diameter.

* See "The Strength of Pillars: An Analysis," by Leopold Eidlitz, Transactions Am. Soc. C. E., vol. xxxv.

"The movement of the pillar head from a horizontal position to one sufficiently inclined to correspond with the stated deflections is so small that the strains generated are inoperative, because the movement is abundantly practicable within the limits of inaccuracy of construction such as exists in practical building.

"If bending were continued to the breaking point, then, no doubt, compound flexure would ensue, but in the absence of loads greater than safe loads, pillars bend with single flexure."

Also, the same author calls attention to the importance of eccentric loading as follows:

"Breaking loads for cast-iron pillars and for wrought-iron pillars, also the respective safe loads, being computed on the assumption that the load is applied in the centre of gravity of the pillar, it is essential that this should be the case accurately, inasmuch as slight deviations cause material differences in their magnitude. A cast-iron pillar 10 ins. in diameter and 11.9 ft. in length (L equal 14.3) will break under a load of 32,000 lbs. per inch metal area, when the load is placed in the centre of gravity of the pillar. When placed 1 in. to one side of the centre it will break under 21,150 lbs., and when placed 0.5 in. off the centre of gravity of the pillar the breaking load is 26,050 lbs. or 19 per cent. less than when exactly in the centre."

"It is also a well-known fact that eccentric loading is under-rated in the absence of a working formula which by one process gives eccentric breaking loads as compared with centric breaking weights and the strength of material.

"These considerations have resulted in the analysis of the strength of pillars, and go to show that safe loads are governed by maximum strain, and not by breaking weights, or else many buildings constructed under the old system would show more serious defects than have been discovered as yet, and also that with a table of safe loads at the command of the engineer or

architect, no eccentric load, no matter how small, should be neglected on the plea that the factor of safety being applied to weights instead of strains covers a multitude of defects not critically examined.''

Every care, therefore, which may be taken in the treatment of eccentric loads will surely add to the capacity of the column, for an eccentric load will necessitate the use of a less mean unit-stress than where the load is applied directly to the centre of gravity of the column section.

Method of Treatment for Eccentric Loads.—As has been previously said, the calculations are extremely tedious and lengthy, but until some more simple and rational formula is devised, eccentric loading should be treated as follows:

(*a*) Determine the section required for the *total* load, both eccentric and concentric, the whole considered as concentric.

(*b*) Find y_1, or half the width of the column.

(*c*) Find the radius of gyration r in the plane of eccentric loading.

(*d*) Find the area of section required to resist the bending moment arising from the eccentric loading, using radius of gyration and y_1 as in the assumed section. The moment due to eccentric loading, M_0, will equal the eccentric load $\times$ its distance of application from the axis of column, and as

$$M_0 = \frac{fI}{y_1} = \frac{fAr^2}{y_1}, \quad \text{we have} \quad A = \frac{M_0 y_1}{fr^2}.$$

(*e*) If this second area can be added to the first assumed area of section without changing the radius of gyration and y_1 materially, it may be done, thus obtaining the total area of section without a new solution.

(*f*) If, however, the radius of gyration and y_1 *are* changed materially, in providing for the new area required, then a new assumed sectional area is taken, radius of gyration and y_1 found for it, the solution proceeding as before.

Such calculations involve the use of the radius of gyration, and complete tables are therefore necessary giving the moments of inertia and radii of gyration for all ordinary column sections of the type employed.

Other designers use Rankine's formula for eccentric loading,*

$$fS = P + P_1\left(1 + \frac{X_0 X_1 S}{I}\right),$$

where $f =$ fibre stress;
$S =$ section required;
$P =$ concentric load;
$P_1 =$ eccentric load;
$X_0 =$ distance from neutral axis to extreme fibre;
$X_1 =$ distance from neutral axis to point of application of eccentric load;
$I =$ the moment of inertia of the section.

Girder Connections; Central Loading.—However carefully or slightly the calculations for eccentric loading may be treated, certain practical considerations at least must be regarded in an attempt to secure the best possible transfer of girder loads, etc., to the centre of gravity of the column section. It is very important that the brackets or girder seats which transmit the girder loads to the columns should be designed with reference to bringing such loads to the centre of the column as soon as possible, and also that the column should be capable of acting as a unit under the application of such loads.

In Fig. 118, showing the connections between girders and the Gray column, it will be seen that the girder loads are not directly transmitted to the column centre, nor can they be in any proper manner, owing to the absence of continuous webs.

* See Rankine's "Applied Mechanics," p. 305.

For short pillars, where flexure may be disregarded, and under *concentric loads only*, this form of column may be satisfactory, but under eccentric loading, or under any transverse stresses, such as wind pressures, this type is decidedly objectionable. The Gray column, as shown in Fig. 118, is composed of four

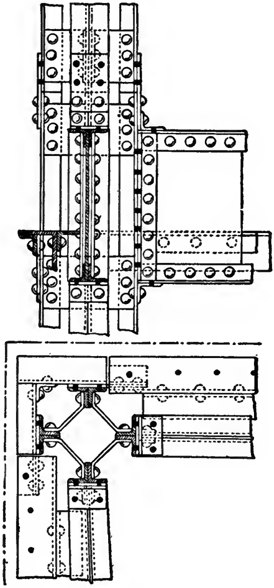


Fig. 118.—Detail of Gray Column and Connecting Girders.

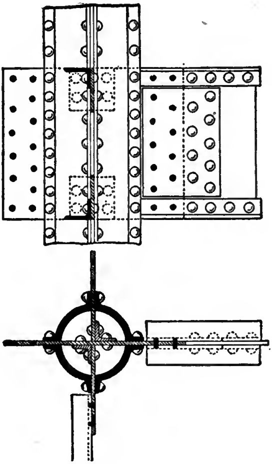


Fig. 119.—Detail of Phœnix Column.

pairs of angles, connected by bent tie-plates which are usually made 8 ins. or 9 ins wide, and spaced 2 ft. 6 ins. centres. These tie-plates cannot transmit either eccentric or transverse loads from the point of application to the several flanges, nor from one flange to the other, owing to the lack of any form of continuous web. The column loads, if eccentric, are borne

mainly by the T-shape to which the girder is connected, and not by the whole column, while transverse stiffness must be measured by the dimensions of the component T-sections, and not by the width of the column itself.

The "standard dimension" Z-bar column is open to the same criticism, as the tie-plates which connect the Z-sections are spaced 3 or 4 ft. centres, thus making the distribution of eccentric or transverse loads purely problematical. The Larimer column has a continuous connection between the two component I-beams, but the connection employed cannot fulfil the office of a continuous web-plate in transmitting the necessary shears. "In other words, no pillar is properly designed unless it has a web which forms a continuous bracing like the web of a girder or truss."

The use of Phœnix columns with "pintle" connections would seem to possess the greatest theoretical advantages under this consideration of central loading. See Fig. 119. This system has been employed under very heavy loading, with pintle-plates over 8 ft. deep. Unless pintle-plates can be used, however, any form of closed column is bad under the consideration of central loading, and here the practical method of loading columns conflicts seriously with the use of an ideal closed section.

The connections of girders to Z-bar columns are better than in most of the forms of closed columns, and even when cover-plates are used this is so (though not in as great a degree), as the column may almost always be turned so that the heavily loaded beam may be introduced between the Z-flanges. This advantage is especially great at the tops of buildings where small columns without cover-plates carry beams with heavy loads, for here the column is open on all four sides, so that all loads may be taken to the centre of the column. The box column of plates and angles, however, possesses this same advantage, though not to as great an

extent in the lighter sections. The possibility of changing the section of a column so that the radius of gyration shall be greater or less in either direction across the section must not be overlooked, for if all the loads occur on one side of a column, it is a great advantage to have the radius of gyration greater in the line of the load.

Convenient Connections — Splices. — These features in column construction are very important ones, and as column splices usually occur at or near the floor-levels, where the connections between the columns and the floor system occur, it is best to consider these two details together.

Satisfactory details can easily be made for almost any of the various column sections, provided continuous column splices are not required, and provided the beams or girders are symmetrically placed and loaded, and all occur at the same elevation; but where irregularity in the girders is necessitated, on account of load, position and elevation, as is almost always the case, and where continuous vertical splices are desired, as should always be secured if possible, it will be found that the various column forms differ widely in their adaptability to these conditions.

It will be seen at a glance that several of the column types are totally unfitted for satisfactory vertical splicing, or else for irregular girder connections, or possibly for both. Thus the Larimer column offers no practical method of splicing except through the use of horizontal cap-plates, while very heavy or irregular girder loads are difficult and oftentimes impossible to support, due to the difficulties experienced in making proper connections to the very limited column surfaces.

The use of cap-plates in Phœnix columns is as shown in Fig. 120, consisting of angles riveted to the extended fillers, on which a plate is placed, holding the girders and the superimposed column. The upper column is held down by angles riveted to the bed-plate. Under eccentric loading a consider-

able tilting movement occurs in this column, unless used with pintle-plates, as before suggested. Connections were made with bent plates in the Old Colony Building, Chicago, as shown in Fig. 121.

In the "World" Building, New York City, 8-section Phœnix columns were employed, with horizontal diaphragms, or bed-plates for splices and girder connections, very similar to

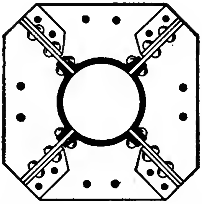


FIG. 120.—Detail of Phœnix Column Splice.

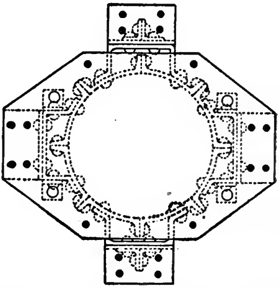


FIG. 121.—Detail of Phœnix Column used in Old Colony Building.

the detail shown in Fig. 120. In the R. G. Dun Building, New York City, 8-section Phœnix columns were spliced by means of vertical diaphragms or pintle-plates, these being in the form of an X, and shop riveted to the lower column section. The ends of the columns were faced, and slots were left between the segments of the upper section to receive the pintle-plates projecting above the joints, the connection being field riveted after bringing into proper position. See Fig. 122. All of these connections are rather complicated, and result in a large amount of work in the preparation of details and in shop labor. Also the necessary changes in the diameters of Phœnix columns for members of different capacities make butt-joints impossible except through the use of horizontal bearing plates.

Before the introduction of vertical splices, Z-bar columns

were generally detailed as shown in Fig. 123, taken from the Monadnock Building, Chicago. The column shafts were placed centrally over one another, with a horizontal cap-plate between (varying from $\frac{5}{8}$ in. to 1 in. in thickness), which was attached to the column shafts by means of upper and lower

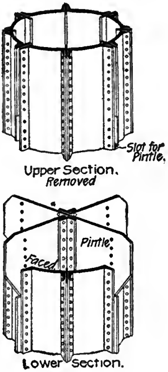


FIG. 122.—Detail of Phœnix Column Splice used in R. G. Dun Building, New York.

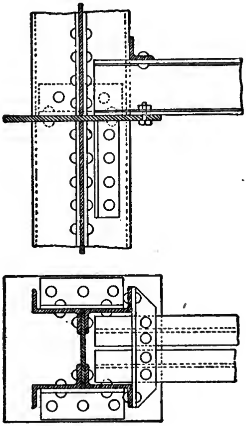


FIG. 123.—Detail of Z-bar Column Splice. Monadnock Building.

connection angles. The girders rested on the cap-plates direct, and if the loads were large, vertical stiffening angles were riveted to the column shaft beneath, to aid in supporting the bed-plate as shown in the illustration. The girders were riveted or bolted through the lower flanges to the bed-plate, and through the upper flanges to a knee attached to the upper column section. Small steel "gibs" or wedges were sometimes dropped in between the top ends of the girders and the column shaft, to take up any possible transverse stresses. If

the girders occurred at different levels, or were of different sizes, cast-iron bolsters were used, resting on the cap-plates.

Single-story lengths of box columns of plates and angles are often detailed as shown in Fig. 124. $\frac{3}{4}$-in. cap-plates are used, with angle-knees connecting to the upper and lower column sections.

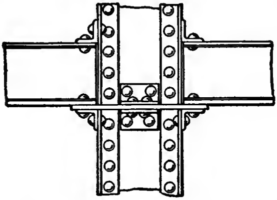


FIG. 124.—Detail of Box-column Splice.

These methods (Figs. 123 and 124) of connecting the tiers of columns together by means of cap-plates and small connection angles, are far from satisfactory, and in good classes of work have been entirely discontinued. Such details may be sufficient to prevent lateral displacement, but because of the bending or elasticity of the horizontal bed-plates and connection angles, and the large ratio of the height of the column to the base, these horizontal splices contribute very little to the rigidity of the structure.

The overturning or lift on the windward side is almost always less than the resistance due to dead weight; but the shear is liable to be overlooked, tending, as it does, to topple over all of the columns of a story. The column connection described is not stiff enough to prevent a slight movement, which can be prevented by wind-bracing only; and, even with wind-bracing, it introduces a weakness of the column at the floor-level, which can largely be obviated by means of continuous columns.

Vertical Column Splices.—In the Masonic Temple, the use of two-storied column lengths was first tried, as an additional factor of stiffness in so high a building, with the joints "staggered," or each column breaking joints with its neighbor. The next step was to discard the bed-plates entirely, using

vertical connection-plates for all column splices. Fig. 125 shows a column splice with connections for the floor-girders and wind-bracing, employed in the Pabst Building, Milwaukee, by S. S. Beman, architect. The floor-girders are made of latticed channels, and the sway-rods are connected to the vertical splice-plates of the columns, much as the laterals in bridge-work are connected to the chords.

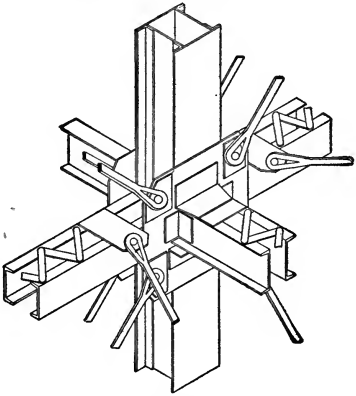


Fig. 125.—Detail of Column Connections and Wind-bracing. Pabst Building, Milwaukee.

The following clauses relating to the splicing of the Gray columns used in the Reliance Building are from the specifications for the steelwork: "The columns will be made in two-story lengths, alternate columns being jointed at each story. The column splice will come above the floor, as shown in the drawings. No cap-plates will be used. The ends of the columns will be faced at right angles to the longitudinal axis of the column, and the greatest care must be used in making this work exact. The columns will be connected, one to the other, by vertical splice-plates, sizes of which, with number of rivets, are shown on the drawings. The holes for these

splice-plates in the bottom of the column shall be punched $\frac{1}{8}$ in. small. After the splice-plates are riveted to the top of the column, the top column shall be put in place and the holes reamed, using the splice-plates as templates. The connections of joists or girders to columns will be standard wherever such joists or girders are at right angles to connecting faces of columns. Where connections are oblique, special or typical details will be shown on the drawings."

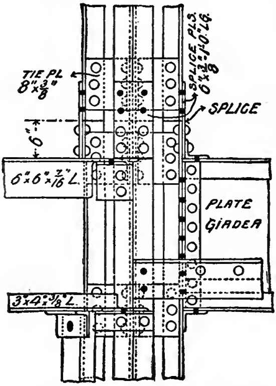


Fig. 126.—Detail of Column Splice. Reliance Building.

Fig. 126 illustrates a typical column splice in the Reliance Building, at a point where the bay-window framing joins the column.

In considering the subject of wind-bracing in the following chapter, it will be seen that rigid connections between the individual columns themselves and between the columns and the floor-girders, contribute an element of resistance of very considerable value to the structure, and this rigidity of the joints is particularly valuable where no special system of trussing is provided to resist the wind strains. If complete vertical splices are used, the columns are made practically continuous, or a unit from foundation to roof, and failure can only occur by breaking or bending; and if such splices are further supplemented by web connections between the columns and the girders, the resultant joint or assemblage of joints will prove as simple as it is efficient. Fig. 127 illustrates these connections, as used in the American Surety Co.'s Building, New York. This figure also shows the connection of the sway bracing.

The necessity of continuity in the columns and web con-

nections for the girders should not be limited to cases in which no additional wind-bracing is provided, nor should efficient wind-bracing be neglected even with these additional factors, as will be pointed out more fully in Chapter VIII.

Advantages, therefore, as regards *convenient connections* and *splices*, or as regards *efficient connections* and *efficient splices*, are found to result principally through the use of rectangular or box-column forms, such as the Z-bar column, or those made of plates and angles or plates and channels. These types present square surfaces for connections with the girders, thus allowing web splices if desired; bracketing for irregularly placed beams can be easily cared for, and continuous vertical splices can generally be arranged for all ordinary cases by introducing fillers where the change in size is not too great. Under these considerations the Z-column with or without cover-plates, and the plate and angle column as shown in Fig. 127, are unsurpassed.

Relation of Size of Section to Small Columns.—It is not generally desirable in building construction to have a very small column in the upper stories, because girder loads are so much heavier, proportionately, than the column loads. Sometimes as many as six beams must connect with an upper-story column at one level, and in such cases it is almost impossible to make good connections with a small column.

It is therefore advisable to use some type of column which, under these conditions, will allow of sufficient size or surfaces to obtain the required connections, and which will still not require excess or waste material through providing such form. In this respect, Z-bar columns are often undesirable, as a minimum size 6-in. column may be insufficient for the girder connections, and any increase in size is attended by a radical increase in weight. A thickness of metal less than $\frac{5}{16}$ in. should never be used, and a minimum thickness of $\frac{3}{8}$ in. is better for conservative practice. Any form, therefore, which calls

for any amount of metal at or near the centre of gravity of the section, such as columns of the I form and Z-bar sections, is undesirable under these conditions, and a form possessing a

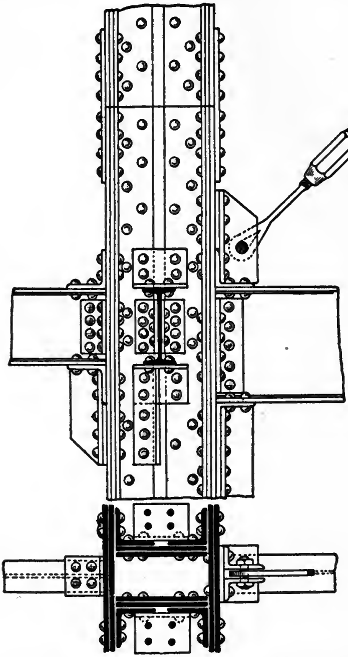


FIG. 127.—Detail of Girder and Column Connections. American Surety Co.'s Building, New York.

large radius of gyration for a minimum of metal will be found preferable.

Fireproofing Capabilities of the Section.—The rectangular column sections will not, of course, fireproof as compactly as the circular sections, but when the room thus lost is used for "pipe-space," as is becoming more and more frequent, this

point has great value in the estimation of architects. In the Columbus Building, Chicago (1893), a square hole was cut in all of the bed-plates of the columns to allow the passage of pipes inside of the column areas. Such a cutting of bed-plates cannot be too severely condemned. The increased use, however, of vertical splices in columns, instead of horizontal bed- and cap-plates, allows all water-, waste-, and vent-pipes to be carried up along the side of the metal columns, and inside the fireproofing slabs, where the room may be had without too much waste. It is not advisable to place any piping inside of the metal columns, and hence such sections as the Phœnix and Keystone Octagonal offer no advantages in this respect. The columns of plates and angles, channels, Zees, and the Gray column, all allow considerable pipe-space within the minimum circular or rectangular enclosure for fireproofing.

It would seem, however, that separate ducts in the walls or along the sides of the columns for all piping would be far better than such concealed risers. Separate ducts would result in increased outlay, but they would offer the great advantage of allowing inspection of all piping whenever and wherever desired.

It sometimes becomes desirable, for architectural effect, to keep the column sizes within very limited areas. Figs. 128 and 129 show two column forms which were used in the Waldorf-Astoria Hotel, New York, where a heavy concentration of metal was required within a minimum circular form to allow the use of enclosing shells of polished stone. The column shown in Fig. 128 was composed of 2 13″ 52-lb. channels, 2 plates $20'' \times \frac{3}{4}''$, 2 plates $12\frac{1}{2}'' \times \frac{3}{4}''$, 2 plates $10'' \times \frac{3}{4}''$, and 4 angles $6'' \times 3\frac{1}{2}'' \times \frac{3}{4}''$. That shown in Fig. 129 was made of 8 angles $5'' \times 3\frac{1}{2}'' \times \frac{11}{16}''$, 2 plates $9'' \times \frac{3}{4}''$, 2 plates $16'' \times \frac{3}{4}''$, and 4 plates $5'' \times \frac{3}{8}''$.

Summary.—From a careful weighing of the foregoing practical considerations in column design, it will be found that

no more satisfactory type can be adopted than the box column of plates and angles. This section has been employed in some

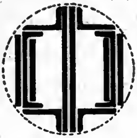


FIG. 128.—Column Section used in Waldorf - Astoria Hotel, New York.

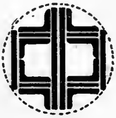


FIG. 129.—Column Section used in Waldorf - Astoria Hotel, New York.

of the heaviest building columns ever designed, and in many of our most important high buildings. Box columns were used in the Masonic Temple in Chicago, as has before been stated, and in the Ivins or Park Row Building, New York. In the latter structure the heaviest column was designed for a load of 2,900,000 lbs., and was composed of 3 web-plates $24'' \times \frac{11}{16}''$, 4 covers $48'' \times \frac{11}{16}''$, and 8 angles $6'' \times 6'' \times \frac{13}{16}''$, as shown in Fig. 130. In the Waldorf-Astoria Hotel a column was used

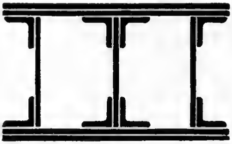


FIG. 130.—Heavy Column Section. Park Row Building, New York.

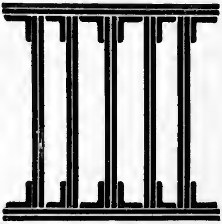


FIG. 131.—Heavy Column Section. Waldorf-Astoria Hotel.

as shown in Fig. 131, this probably constituting the heaviest pillar ever used in building construction. This was required to support the large trusses over the ball-room in the first story. The load carried was estimated at 5,400,000 lbs., and to obtain the required sectional area, 10 web-plates, 4 covers, and 12

angles were used. The length was 30 ft. 4 ins., and the weight of the column was 46,980 lbs.

Columns made of channels and plates, and the standard Z-bar columns, rank next in order to those made of plates and angles. Channel columns are more limited as to size and area, through the use of the component channel members, while very heavy Z-bar columns become rather complicated in form, as shown in types 3 and 4 of Fig. 113.

The largest Z-column section in "The Fair" Building, Chicago, consists of 4 Z-bars $6'' \times \frac{7}{8}''$, 2 webs $16'' \times \frac{3}{4}''$, 6 covers $16'' \times \frac{13}{16}''$, aggregating an area of 142 sq. ins. and carrying a load of 1,700,000 lbs. The largest Z-column in the new Y. M. C. A. Building, Chicago (see Fig. 132), was a two-story column 24 ft. 3 ins. long, composed as follows: 4 Z's $6'' \times 3'' \times \frac{7}{8}''$, 2 plates $24'' \times \frac{7}{8}''$, 2 plates $16'' \times \frac{7}{8}''$, 1 plate $14'' \times \frac{3}{4}''$, 2 plates $26'' \times \frac{7}{8}''$, 4 angles $4'' \times 4'' \times \frac{13}{16}''$, 4 angles $5'' \times 4'' \times \frac{7}{8}''$ — total = 218 sq. ins.

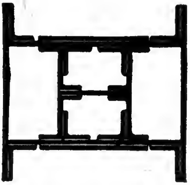


FIG. 132.—Heavy Column Section. Y. M. C. A. Building, Chicago.

Column Bases: Cast Plates.—As the concentrated column loads must be distributed over the foundations which receive them, some form of distributing base or shoe is necessary at the bearing ends of all the lowest tier columns. The area and character of such bases are determined by the amount of the column load, and by the allowable pressure per square inch on the underlying foundation. If the shoe or base is to rest on grillage beams, or on some special form of distributing or cantilever girder, at least one dimension of the base is usually fixed by such conditions; but if the bearing is to be upon concrete, brick masonry, or dimension stones, the required area of the base is determined by dividing the total column load by the allowable pressure per square inch on the foundation material. The quotient will give the required number of

square inches in the area of the base. The unit pressure on various classes of foundation materials are usually fixed by the municipal building laws. If timber grillage is to receive the column base, distributing beams or girders will usually be required between the base and the grillage, in order to obtain the requisite bearing areas.

When the column loads are small, solid cast-iron base plates may be employed. These are usually cast with a bevel or wash, and the column may be secured by means of tap-bolts through small knees attached to the column shaft, or by means of bolts passing through ribs or flanges cast on the plate, as shown in Fig. 133. Such base plates are proportioned as follows:

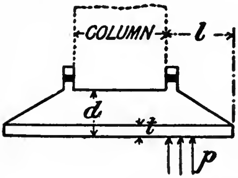


FIG. 133.—Cast-iron Base Plate

The bottom or bearing area of the plate is determined by dividing the total column load by the allowable bearing per square inch, p, upon the material which supports the base. The size of the plate thus found, and the known size of the column, will then fix the projection l. To determine the thickness d, let

$M =$ bending moment in inch-pounds, $= fS$,

where $f =$ allowable extreme fibre stress per square inch for tension $=$ 3,500 lbs. to 4,000 lbs.;

$S =$ section modulus.

Then, for the projecting portion of casting,

$$M = \frac{Pl}{2}, \text{ where } P = pl.$$

Hence $M = \dfrac{pl^2}{2}$.

But $S = \dfrac{bd^2}{6}$, where $b = 1$ for a section 1 in. wide.

Hence, as $M = fS$

$$\frac{fd^2}{6} = \frac{pl^2}{2}, \quad \text{and} \quad d^2 = \frac{3pl^2}{f},$$

or

$$d^2 = l^2 \frac{3p}{f}, \quad \text{or} \quad d = l \sqrt{\frac{3p}{f}}.$$

The thickness t is usually made about equal to $\frac{d}{4}$.

Steel Column Shoes.—For heavier column loads, where simple cast plates as above are not sufficiently strong to act as distributors, steel shoes or cast-iron bases or column stands will be required.

Built-up shoes of steel plates and angles are considered by many to distribute the loads more efficiently over rectangular base areas than results from the use of cast-iron bases. The latter, however, are much more common than the steel bases.

Steel column shoes may be calculated as follows:

Referring to Fig. 134, assume a 12-in. Z-bar column, carrying a load of 344,000 lbs. The bed-plate or shoe is to rest on a grillage foundation, the top layer of which is composed of 4 15-in. 42-lb. I-beams. Hence the shoe is made 24 ins. wide to span these I's.

The column shaft carries one-fourth of the total load directly to each of the two central beams, thus leaving the shoe to transmit one-fourth of the load to each of the two outer beams. The total load being 344,000 lbs., the amount transmitted by the shoe on either side is $\frac{344,000}{4} =$ 86,000 lbs., or 43,000 lbs. at each flange of the column.

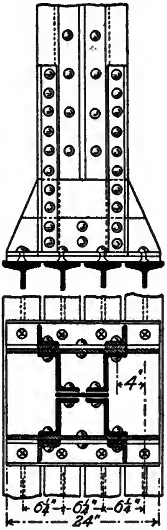


FIG. 134.—Steel Column Shoe.

The horizontal distance from the line of vertical rivets in the column shaft to the centre line of an outer beam is 4 ins., hence the bending moment

$$M = 43,000 \times 4 = 172,000 \text{ in.-lbs.}$$

The section modulus,

$$S = \frac{M}{f} = \frac{172,000}{16,000} = 10.75.$$

But $S = \frac{bh^2}{6}$, where b equals the thickness of the gusset plate, and h the depth of the gusset. Assuming a gusset plate 12 ins. deep, we have

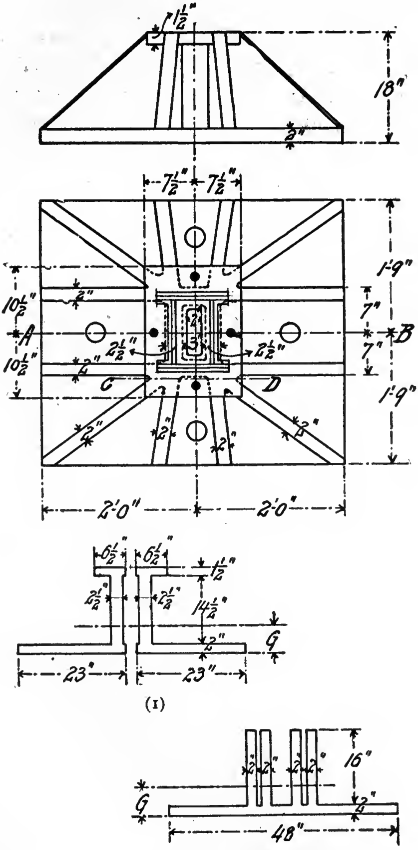
$$10.75 = \frac{b \times 144}{6}, \text{ whence } b = .448 \text{ ins.}$$

The gussets should therefore be ½ in. thick. For the vertical lines of rivets transmitting the loads from the column shaft to the gussets, each row must transmit 43,000 lbs. at single shear. The value of a ¾-in. rivet in single shear at 10,000 lbs. per sq. in. is 4,420 lbs. Hence the number of rivets required in each line is $\frac{43,000}{4,420} = 10$. The gusset assumed is not deep enough to take so many rivets, so that it becomes necessary to use vertical stiffeners as shown in Fig. 134, these being "milled" or planed at the bottom to bear upon the bottom angle-foot, and extended up the column shaft a sufficient distance to take six rivets above the four in the gusset.

Cast-iron Column Bases.—The most ordinary form of distributor for column loads over foundation areas is the cast-iron column stand or base, also called shoe and stool, though shoes are generally taken to mean steel bases as previously described. The proportioning or calculations for cast bases vary considerably in actual practice—indeed, it is more than probable that a very large proportion of cast bases are never figured at all; but, even when figured according to one method, any particular casting will be found to vary very considerably if figured by other methods in more or less common use.

In designing base castings, the following elements of the problem are fixed by the conditions of the foundation: the

column load, the size of column, and the character of foundation which is to receive the base. The column load and the



(2)

FIG. 135.—Cast-iron Column Stand.

foundation material will at once fix the area of the base, and hence the actual load per square inch reacting on such base. The height of base casting may be taken at from one-third to

one-half the side, and for such column loads as may not be safely supported by solid base plates, a minimum thickness of metal should be $1\frac{1}{4}$ ins. for the top and bottom plates and for the ribs. The dimensions of the top plate are determined by the column size, care being taken to provide connection holes for bolts connecting the base casting to flanges riveted to the column.

With these conditions fixed, a trial section may be assumed, and calculated as follows:

Referring to Fig. 135, assume a column-load of 980,000 lbs., a column section as shown, and a cast base 48 ins. by 42 ins. The load per square inch on the bottom plate is then $\frac{980,000}{2,016}$ or 486 lbs.

The resultant moment of the external forces acting on the casting must equal $\frac{fI}{y_1}$,

where f equals the allowable extreme fibre strain;

I equals the moment of inertia of the section;

and y_1 equals the distance of the extreme fibres from neutral axis of section.

To find I, consider a section of the casting on the line AB (see diagram 1). The position of the neutral axis must first be found, and the distance G of the neutral axis from the bottom of section will equal

$$\frac{\Sigma ag}{A} = \frac{19\frac{1}{2} \times 17\frac{1}{4} + 72\frac{1}{2} \times 9\frac{1}{4} + 92 \times 1}{19\frac{1}{2} + 72\frac{1}{2} + 92}$$
$$= 5.918 \text{ ins., or call } 6 \text{ ins.}$$

The moment of inertia of the entire section, I, will equal the sum of the moments of inertia of the various rectangles, I'', etc., plus the areas of the several rectangles multiplied by the squares of the distances from their individual centres of gravity to the neutral axis of entire section. Or,

$$I = \Sigma(I'' + Ad^2),$$

and remembering that I'' for a rectangle is $\frac{bh^3}{12}$, we find I to equal 6,797 in.-lbs.

Then, taking f for tension at 4,000 lbs.,

$$\frac{fI}{y_1} = \frac{4{,}000 \times 6{,}797}{6} = 4{,}530{,}000 \text{ in.-lbs.}$$

To find the resultant moment of the external forces for the portion of the casting on either side of the centre line, AB, we have two forces—one, the total pressure on a half of the base, or $\frac{980{,}000}{2}$, which is applied at a point midway between the centre line AB and the edge of base-plate; second, one-half the column-load acting on the top flange of casting, the point of application of which may be taken at the centre of gravity of one-half the column section. Computing this, by the same method as previously used, we find the centre of gravity of the half column section to be 4 ins. from the centre line of column, or line AB.

M therefore equals

$$\frac{980{,}000}{2} \times (10\tfrac{1}{2} - 4),$$

or 3,185,000 in.-lbs., and as this is considerably less than the value of $\frac{fI}{y_1}$ previously found, the casting is amply strong.

For a calculation of the ribs, consider a section on the line CD as in diagram 2.

The neutral axis, computed as before, is found to be 6 ins. up from the bottom of section.

I, calculated as previously, equals 7,210; hence

$$\frac{fI}{y_1} = \frac{4{,}000 \times 7{,}210}{6} = 4{,}806{,}666 \text{ in.-lbs.}$$

The moment on the area of base supported by the ribs in question will equal

$$(486 \times 14 \times 48) \times 7, \quad \text{or} \quad 2{,}286{,}144 \text{ in.-lbs.};$$

hence the ribs might be taken of a lighter section; but due judgment must be exercised to produce a casting of about right proportions, and when possible internal stresses are considered,

and the practically indeterminate solution for a base-plate of this design, it is best to err largely on the safe side in all calculations.

Column-loads.—If the building laws under which the designer is working allow a reduction in the live-loads assumed to be carried by the columns themselves, as in the New York Building Code, or a reduction in, or total disregard of, the live-loads upon the foundations, as is the case in the Chicago ordinance, the dead- and live-loads on all columns should be kept separate to allow the proporitoning of the columns themselves, or of the foundations. Several examples of the reduction of live-loads upon the columns or footings, or both, were given in Chapter IV, but whatever the requirements of the building laws in force, the column-loads are best tabulated by means of "column-sheets." These vary considerably in form and completeness, according to the refinement with which the various classes of loads are treated.

Column-loads include floor- and roof-loads, wind-loads, spandrel- and pier-loads, the weights of the columns themselves and their fireproof coverings, and special loads such as tanks, vaults, safes, elevator-loads, and any permanent machinery, the latter class of loads being usually treated as concentrated. Floor- and roof-loads can readily be taken from the floor plans, provided the end reactions of all floor-girders are marked on the original drawings, as the girders are calculated. Wind-loads are determined as explained in Chapter VIII, while pier- and spandrel-loads are calculated as described in Chapters V and VI.

Column-sheets.—As soon as all loads in the structure have been definitely settled, the column-sheets may be started, thus forming a tabulated list of all the loads transferred to the footings through the columns. From these sheets may be seen the approximate load that each column must carry at any floor, starting with the upper-story columns, supporting the roof-load only, and adding in the loads at the successive floors

down to the foundations. The column weight itself is first assumed, and then corrected, after the proper section is obtained.

The column-sheet used in the Masonic Temple calculations was as follows:

		Column 1.		Column 2.	
		Load on Column.	Load on Footing.	Load on Column.	Load on Footing.
Roof.	Floor load.............				
	Masonry piers.........				
	Spandrels.............				
	Elevators.............				
	Tank loads............				
	Weight of column......				
	Total..............				
20th Floor.					

The column-sheet used in the Venetian Building was made as in the accompanying table:

Column 1.	Roof.	Attic.	12th Floor.		Basement.	Total.
Load from column above........						
Floor load, dead................						
Floor load, live.................						
Spandrels......................						
Elevator loads..................						
Estimated weight of column......						
Total........						
Wind Loads.						
Concentric wind loads...........						
Eccentric wind loads............						
Total wind load............						
Column 2.						
Etc.						

The following column-sheet is to be recommended as combining all requisites in a tabulated statement:

		Column 1.			Column 2.
		Load on Column. Concentric.	Load on Column. Eccentric.	Load on Footing.	
Roof.	Floor load.............				
	Masonry piers.........				
	Spandrels.				
	Elevator loads.........				
	Tank loads, etc........				
	Weight of column......				
	Wind				
	Total.....				
	Area required for col...	sq. in.	sq. in.	Foot'g area	
	Material of column.....			sq. ft.	
		Load on Column. Concentric.	Load on Column. Eccentric.	Load on Footing.	
16th Floor.	Floor load............ Etc.				

The final loads on the basement columns taken from these sheets will show the loads for which the footings themselves must be figured, while the final loads on the footings will give the weights for which the clay areas must be proportioned, if the foundations are on yielding soil.

The following table shows the column-sheet loads for a few of the columns in the Fisher Building, Chicago.* The assumed roof- and floor-loads for the same building, and their distribu-

* See E. C. Shankland in Minutes of the Proceedings of the Institution of C. E., vol. cxxviii.

tion on joists, girders, columns and footings, were given in Chapter IV.

		No. 4.	Nos. 9–12–21 10–19–22 11–20–23	Nos. 28–31 29–32 30–33	Nos. 34 35
		lbs.	lbs.	lbs.	lbs.
Attic.	Roof	10,500	10,230	14,190	21,450
	Column and casing	1,500	....	3,000	3,000
	Tanks				
	Elevators	18,000			
	Total	30,000	10,230	17,190	24,450
18	Floor	11,780	17,670	24,510	37,050
	Column and casing	2,440	....	4,870	4,870
	Cornice, etc.	78,750	30,000	....	....
	Tanks	....	1,700	28,900	5,000
	Total	122,970	59,600	75,470	71,370
17	Floor	15,500	23,250	32,250	48,750
	Column and casing	2,440	8,230	4,870	4,870
	Spandrel	21,580	13,050		
	Total	162,490	104,130	112,590	124.990
1	Total	711,090	694,530	628,130	865,590
	Floor	14,260	21,390	29,670	44,850
	Column and casing	2,840	12,000	5,670	5,670
	Spandrel-mullion	19,600	8,820		
	Total	747,790	736,740	663,470	916,110
Basement.	Floor	16,120	24,180	33,540	50,700
	Column	2,000	10,200	4,000	4,000
	Sidewalk	....	10,660		
	Party-wall	26,780			
	Total	792,690	781,780	701,010	970,810
Footing.	Live-load, deduct	34,100	51,150	70,950	107,250
	Footing	758,590	730,630	630,060	863 560

Proportioning Column-sizes.—There have been few experiments of value on the ultimate strength of full-sized steel columns of the types in more ordinary use. Building operations have to be conducted too quickly to allow many tests on the full-sized columns before using. Tests have been made on full-sized Gray columns, and also on the Larimer column, but

these are both special shapes, and the tests have little bearing upon data regarding the more common forms. The only full-sized tests on Z-bar columns were made by C. L. Strobel, then Chief Engineer of the Keystone Bridge Company, (see Transactions of the American Society of Civil Engineers, April, 1888), who introduced this shape into the United States. But even these tests are hardly fair ones for present comparisons, as lattice bars were used instead of web plates, and almost all the tests were for a much higher ratio of the radius of gyration to the length of column than is ordinarily met with in building work. The tests were also for iron columns, and not for steel. It seems as though higher breaking loads would be obtained for the majority of steel columns as used at the present time. Burr, in his "Strength and Resistance of Materials," deduces formulæ for the Keystone and Phœnix columns, but none for the Z-column or the box column of plates and angles. The latter type was used in the Masonic Temple in two-story lengths, lattice bars being used instead of plates in the lighter columns. But as the height of a single story was less than 12 ft. unsupported length, a uniform unit-stress of 12,500 lbs. per sq. in. was used without reduction by the radius of gyration, for all concentric loading. Columns with eccentric loads were figured for a unit-stress of 12,500 lbs. per sq. in., reduced by Rankine's formula for eccentric loading.

For columns of ordinary single-story lengths, this practice of proportioning the section by simply dividing the total column-load by the allowable stress per square inch, will serve all practical requirements, as previously explained in the discussion of Gordon's formula.

In the Venetian Building the columns without strains from wind-bracing were figured at 15,000 lbs. per sq. in. for all concentric dead- and live-loads, with an extra allowance for eccentric loads. The columns carrying strains from the wind-bracing were figured at 20,000 lbs. per sq. in. for all concentric

loads,—dead, live, and wind,—with an additional allowance for eccentric loading. In these columns the wind-strains amounted to from 35 to 40 per cent. of the total load, so that this mode of treatment of using a higher unit-stress gave a much greater section to the column than if a lower unit-stress had been used and the wind forces disregarded. These unit-stresses have been used in a number of high buildings, notwithstanding some rather severe criticism.

In "The Fair" Building (W. L. B. Jenney, architect), 12,000 lbs. was used uniformly on all columns, with no allowance for eccentric loading. This building is one of the heaviest in the city of Chicago, being figured for 130 lbs. live-load per square foot for the 1st, 2d, 3d, 4th, and 6th floors, 200 lbs. for the 5th floor, 100 lbs. for the 7th and 8th floors, with the rest at 75 lbs., all in addition to dead-loads. Great care was taken in providing good connections throughout.

In the Fort Dearborn Building, by the same architect, a unifoım unit-stress of 13,000 lbs. per sq. in. was used on all columns, made of channels and plates, with a proper reduction for eccentric loading.

The writer believes that with the use of a mild steel, of an ultimate strength of from 65,000 to 68,000 lbs. per sq. in., 15,000 or 16,000 lbs. per sq. in. may safely be used for all concentric dead-, live-, and wind-loads combined (with an additional allowance for eccentric loading as before described), provided that the wind-pressure is taken at not less than 30 lbs. per sq. ft., and that the live-loads on the floor systems are assumed as required by the municipal building laws. With careful regard for all connections, and remembering that the strength of a structure lies in its weakest point, these unit-stresses would seem to satisfy both the conditions of proper economy and satisfactory design.

The use of 20,000 lbs. per sq. in., as in the Venetian Building, would seem too high, especially when the live-load

is but 35 lbs. per sq. ft. on the floor systems, and when but 50 per cent. of this is considered as transferred to the columns.

In columns of long length, or for lengths of 90 radii and over, calculation by the radius of gyration becomes necessary. For such cases the standard formula,

$$p = 17{,}100 - 57\frac{l}{r},$$

may be used, where p equals the allowable stress per square inch, l equals the length in inches, and r equals the radius of gyration of section in inches.

This formula is derived from the tests on full-sized Z-bar columns before referred to, and gives values about 20 per cent. in excess of those found to be true for the iron columns tested.

Modern building design has rapidly developed the necessity for columns of extraordinary lengths and areas. In the Schiller Theatre Building, Chicago, Phœnix columns were used in connection with the trusses over the auditorium of a length of 92 ft. 10 ins., weighing 25,000 lbs. each, while in the Chicago Board of Trade, 12-section Phœnix columns, 3 ft. 3 ins. in diameter, were employed for an unsupported length of 90 ft. The large columns in the Waldorf-Astoria Hotel were previously mentioned.

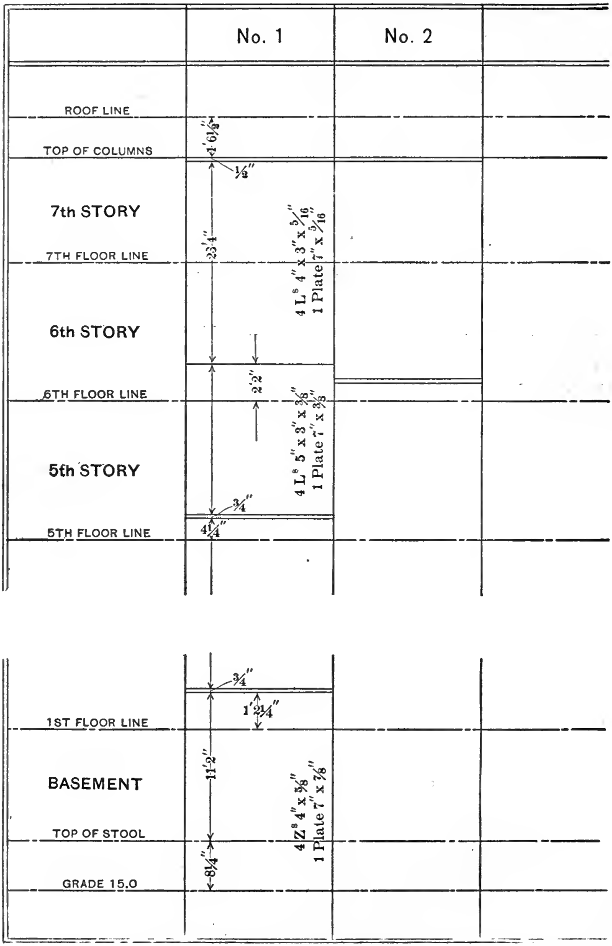
In proportioning the sizes of material for columns of two-story lengths, no change in section need be made provided the difference in loads is slight. It will often be more economical to proportion the member for the heavier load, and to let the required section continue uniform, rather than to decrease the section slightly and thus cause the splicing or rearrangement of material. If the difference in loads is considerable for a two-story length, additional cover-plates may be riveted on to the lower story length only, thus having one cover-plate below and none above, or one continuous cover for the two stories and an additional one in the lower section. If channel columns

are used, the same size and weight of channel section may be used for the entire length, making the difference in area in the thickness of the flange-plates, or the same sized flange-plates may be used throughout, by changing the weights of the channels in each story.

A convenient schedule for column lengths, splices, and material, may be made as shown on page 242.

Column Details and Splices.—The details or shop drawings for columns must show the required connections or shelf-angles for the various beams and girders attaching to the column, the spacing of the shaft-rivets and latticing or tie-plates if required, besides the details of splices. Fig. 136 illustrates a shop drawing of a three-story Z-bar column, with the various girder connections and splices.

The details employed in the design of shelf-angles or supports for girders carried by the columns will vary considerably with the different types of columns, but in general it may be stated that where a sufficient number of rivets cannot be obtained direct through the shelf-angle into the column-shaft, the additional rivets must be secured by means of stiffening angles. Assuming a safe shearing-stress of 10,000 lbs. per sq. in. for rivets, and $\frac{3}{4}$ in. diameter rivets as are usually employed except for the heaviest work, the value of each rivet in single shear is 4,420 lbs. This may be used for all metal $\frac{5}{16}$ in. thick or over, but for $\frac{1}{4}$-in. metal the lesser bearing value of 3,750 lbs. would have to be used. Taking, then, the shelf-angle shown in Fig. 137, the safe load would be four times 4,420 lbs., or 8.8 tons. If the end reaction of the girder to be supported is greater than this, stiffening angles must be introduced to provide the additional number of rivets, as in Fig. 138, where the safe load becomes double the former, or 17.7 tons. The stiffening angles are placed directly against the vertical flange of the shelf-angle, and fillers are inserted from the bottom of the shelf-angle to the lower ends of the



Form of Schedule for Column Lengths and Column Material.

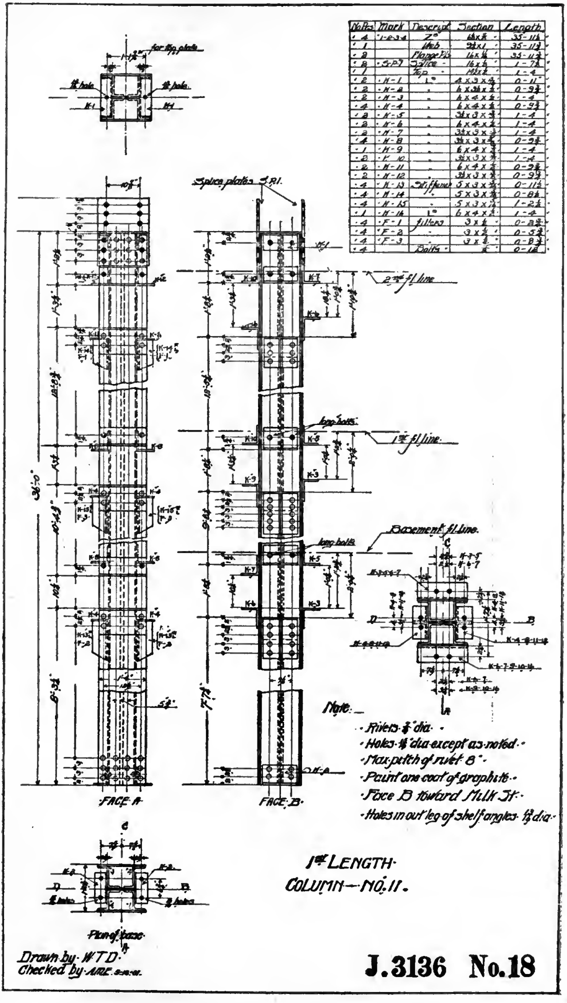


FIG. 136.—SHOP DETAIL OF Z-BAR COLUMN.

stiffeners. Where large loads are to be carried, the upper ends of the stiffeners should be "faced" or planed, to insure a full bearing for the seat.

Column sections are ordinarily increased from story to story by using increasing thicknesses of shapes of the same general sizes, or by the addition of reinforcing cover-plates, etc. In such cases the variations in the principal dimensions of the

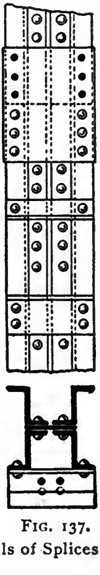


FIG. 137.

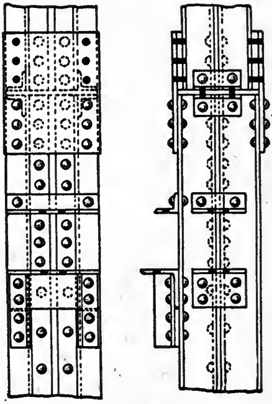


FIG. 138.

Details of Splices and Beam Connections for Z-bar Columns.

cross-section are very slight, and the splices may be made by butt-joints, thus utilizing the direct bearing of the upper member upon the lower one. The splice-plates are therefore not required for transferring any vertical load, but as shear, wind-strains, and rigidity are to be provided for, the splice-plates must be designed accordingly, and good practice has made three lines of rivets above and below the joint a mini-

mum. Fig. 137 illustrates a column splice for the Z-section where the size of the column changes only in the thickness of the material. The slight variations usually found between the two sections may be made up in filler plates inserted between the Z-flanges and the splice-plates.

If a radical change is made in the dimensions of the cross-section, as, for instance, in changing from a 10-in. Z-column to an 8-in. Z-section, a horizontal cap-plate or diaphragm must be inserted, as in Fig. 138. This is usually riveted to the lower column, and serves to provide a bearing and distributing surface for the upper column. But as explained previously, continuous vertical splices are far preferable for many reasons.

If direct bearing between the columns cannot be utilized, on account of differences in the cross-sections, and it is not desired to use horizontal cap-plates, vertical splice-plates may be arranged to transfer the load, in which case the number of rivets must be proportioned to transmit the entire strain.

Column splices are generally made just above the floor-beam connections, this being largely for aid in erection as before mentioned. It is now customary to "stagger" the splices in adjacent columns, so that if one column splices at the tenth and twelfth floors, those on either side should splice at the eleventh and thirteenth floors. Column splices should *always* be riveted, never bolted.

Column ends should always be "faced" or "milled" to a true surface which is exactly normal to the column axis. The finished length from end to end must be *exact*, and the member should be free from bends or buckels.

Fireproofing of Columns.—As the columns carry the greatest loads found in modern buildings (some over 3,000,000 lbs.), the proper fireproofing of these members becomes a most important subject for consideration. In only too many cases, however, is this slighted even to a very dangerous

extent, as was proven by the Athletic Club Building fire, before referred to.

The first attempts at making fireproof columns were through the use of a double column, one inside the other, with the intervening space filled with plaster. This idea was patented, and reference may still be found to such construction in the New York building law, as: "The said column or columns shall be either constructed double, that is, an outer and an inner column, the inner alone to be of sufficient strength to sustain safely the weight to be imposed thereon."

The scientific fireproofing of columns by means of terra-cotta was started by Mr. P. B. Wight in 1874, and the Chicago Club house, designed by Treat & Foltz, architects, was the first instance where terra-cotta gores were used around columns. Many systems have since been introduced, and both the hard tile and the porous tile have been used extensively. The cheapest method has been through the use of shells of hard terra-cotta surrounding the column, but not backed up to the metal-work. This system is decidedly faulty in placing so much reliance in the joints alone for stability, as the blocks are simply cemented to one another, and not to the metal column.

The requirements in the adequate fireproofing of columns are:

1. The material must be indestructible by fire and water.

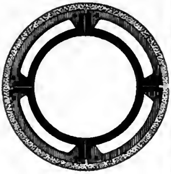
2. The material must be non-heat-conducting.

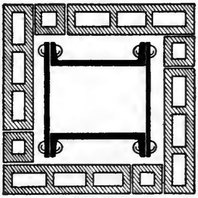
3. The material must be so secured to the column that it cannot be dislodged.

The use of hard fire-clay tiles is only to be recommended when such tiles are hollow, with a proper air-space around the metal column, and even then experience seems to show that the hard tile is in no way as satisfactory under great heat as the more porous kinds. Applications of cold water in combination with heat have also proved the hard tile far less reliable in case of

conflagration than the porous tile. The hard tile is very apt to crack off under such conditions, as has been stated in chapter IV.

The use of hollow blocks of porous tile, well bedded against the metal column, has proved to be the most rigid and efficient. Here, as in terra-cotta floor-arches, the competition





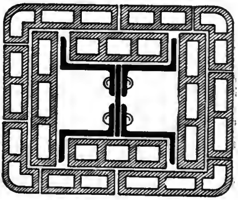


FIG. 139. FIG. 140 FIG. 141.

FIG. 139.—Method of Fireproofing Phœnix Columns.
FIG. 140.—Method of Fireproofing Channel Columns.
FIG. 141.—Method of Fireproofing Z-bar Columns.

in price, which places the better article or method at a disadvantage, is to be deplored. Loosely drawn specifications are also responsible in a great measure for many very common

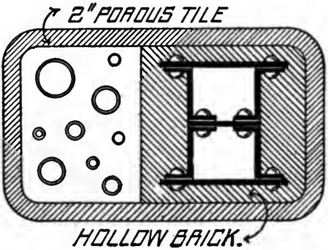


FIG. 142.—Method of Fireproofing Columns, Monadnock Building.

defects. Figs. 139, 140, and 141 show the ordinary methods of placing the fireproof furring for columns.

The Z-bar columns in the newer portion of the Monadnock Building were fireproofed as shown in Fig. 142 up to and

including the eighth floor. Hollow bricks, laid in cement mortar, were built solidly around the columns to a line distant 4 ins. from the extreme points of the metal-work, and a 2-in. coating of hollow tile was then laid against the brick backing extending beyond the column in one direction, to serve as a space for vertical pipes. The columns above the eighth floor received the hollow-tile protection only.

For more extended data regarding the use of metal lath and plaster, concrete, and terra-cotta as fireproof coverings for columns, the reader is referred to the chapter on Column Fireproofing in the author's "Fireproofing of Steel Buildings."

Building Laws: Fireproofing.—The requirements for fireproofing the interior columns of office buildings are thus defined by the Chicago ordinance:

"The coverings for columns shall be, if of brick, not less than 8 ins. thick; if of hollow tile, one covering at least 2½ ins. thick. If the fireproof covering is made of porous terra-cotta, it shall be at least 2 ins. thick. Whether hollow tile or porous terra-cotta is used, the courses shall be so anchored and bonded together as to form an independent and stable structure."

"In all cases there shall be on the outside of the tiles a covering of plastering with Portland cement or of other mortar of equal hardness and efficiency when set."

Two layers of any covering made of plastering on metallic lath are also allowed by this ordinance in office buildings.

The New York law requires that columns in fireproof buildings "shall be protected with not less than 2 ins. of fireproof material, securely applied."

CHAPTER VIII.

WIND-BRACING.

A CAREFUL comparison of the treatment of wind forces as applied to the mercantile buildings of to-day leads one to the conclusion that the designers differ very materially in regard to the forces to be resisted, the strength of the materials employed, and the most efficient details of construction. Indeed, there are very many well-known buildings from ten to sixteen stories high that possess absolutely no metallic sway-bracing, and others, scarcely better, where sway-rods, as wind-laterals, were attached to pins through lugs on the cast columns, which lugs were of an ultimate strength of, perhaps, 25 per cent. of the rods. H. H. Quimby, in his paper on "Wind-bracing in High Buildings," * mentions the case of an office building in New York, of seventeen stories, or 200 ft. in height, and 60 ft. wide; 13-in. walls were used front and back, broken by windows and bay-windows, with wind-bracing consisting solely of the interior partitions of 8-in. box tile, with four ribs of $\frac{5}{8}$ in. each, or $2\frac{1}{2}$ in. thickness of tile in each partition. This building towers above its neighbors of five or six stories only, while but a few blocks away is one of seventeen stories, also, but 150 ft. wide, or $2\frac{1}{2}$ times the width of the former, with sway-bracing consisting of 15-in. cannel-struts and 6-in. eye-bars. Such is the diversity of practice.

Some architects have depended solely upon partitions of hollow tile for the lateral stability of their buildings, weak as

* Trans. A. S. C. E., vol. xxvii. No. 3.

the partitions must be through the introduction of numerous doors and office lights. This method of filling in the rectangles of the frame by light partitions *may* be efficient wind-bracing, but the best practice would certainly indicate that it cannot be relied upon, or even vaguely estimated.

A building with a well-constructed iron frame should be safe if provided with brick partitions, and if the base is a large proportion of, or equal to the height, or if the exterior of the iron framework is covered with well-built masonry walls of sufficient thickness; for the rigidity of solid walls would exceed that of a braced frame to such an extent that, were the building to sway sufficiently to bring the bracing-rods into play, the walls would be damaged before the rods could be brought into action.

Hence the stability must depend entirely either on the masonry or on the iron framing; and in veneer buildings, which are being considered here in particular, the latter system of bracing the metal-work must be used, with the walls as light as possible, simply enclosing the building against climatic and injurious forces. This practice has been adopted quite uniformly by all conservative architects and engineers, and will alone be considered here as a method of wind-bracing.

Each building offers its own peculiar conditions to the carrying out of proper wind-bracing, and many factors must be considered for a judicious solution. The height, width, shape, and exposure of the structure, as well as the character of the enclosing walls, will determine the amount of the wind pressure to be cared for, while the details of construction, the internal appearance, and the planning of the various floors will largely influence the manner in which this bracing is to be treated. The architectural planning of the offices, rooms, and corridors often raises most serious obstacles to a proper arrangement of wind-bracing, and the engineer is frequently called upon to make most generous concessions in favor of

doors, windows, passages, and even whole areas, as is sometimes demanded in banking- or assembly-rooms and the like. Such considerations have led to the development of the portal type of wind-bracing. As more and more of the constructional work of large buildings is placed in the care of the engineer, as opposed to the purely architectural or decorative draughtsman, just so will the former insist that a proper regard for construction is of equal value with the artistic portion of the work. The one must supplement the other, instead of giving way to irrationalities of design.

Intensity of Wind Pressure.—The intensity of wind pressure which should be calculated for in the design of high buildings varies greatly, as has before been stated, according to the ideas of the designer. Many architects and engineers are content to provide for a very moderate average wind pressure, on the assumption that extreme pressures are of very rare occurrence, and of very short duration. Other architects, and probably most conservative engineers, believe that it is precisely the unusual and unlooked for emergency which should be foreseen, and that any such additional security should be considered not as a useless waste of expenditure, but in the nature of insurance upon the life and efficiency of the structure.

Statistics as to severe storms or tornadoes show that such extreme conditions are of too frequent occurrence to be ignored without assuming considerable hazard. The reports of the U. S. Signal Service show that between the years 1889 and 1896 great tornadoes averaged about three per annum, the total property loss being about $24,000,000. The most destructive storms were those in Kansas City in 1886, Louisville in 1890, Little Rock in 1894, and St. Louis in 1896.

The relation between the velocity of wind and the pressure exerted upon surfaces normal to its direction is usually expressed by the formula $P = cV^2$, where P equals the pressure in pounds per square foot, c equals a constant, and V

equals the wind velocity in miles per hour. The value of the constant c, depending upon experiment, has been variously computed by different authorities. Some experiments have indicated a value as low as 0.003, others place it at 0.005, while the United States Weather Bureau has adopted the value of 0.004, thus making the formula $P = 0.004 V^2$. The experiments made to determine this value were through the use of gauges with surfaces of 4 and 9 sq. ft. According to this formula, an assumed pressure of 40 lbs. per sq. ft. would correspond with a velocity of 100 miles per hour.

Experiments made at the Forth bridge on two wind-gauges of 300 sq. ft. and 1½ sq. ft. respectively, indicated that with an increase in area the unit of pressure decreased in a very marked degree; but regardless of experiments with gauges, there is sufficient evidence to show that high wind pressures are exerted over far wider areas than is generally supposed.

The extreme velocity in the St. Louis tornado was 120 miles per hour. The greatest wind velocity ever recorded in New York City was 75 miles per hour, for a duration of two minutes. This was recorded by the instruments of the U. S. Signal Service on March 28, 1895, and, according to the formula previously given, is equivalent to a pressure of 22½ lbs. per sq. ft.

A unit of 30 lbs. should serve as a minimum in high buildings of veneer construction. Mr. Quimby, in the paper before alluded to, favors provision for a 40-lb. pressure, with steel bracing strained not over one-third of the ultimate strength; while others, in a discussion of the article, advocate the use of 30 lbs. Mr. Guy B. Waite, M. Am. Soc. C. E., states that "After consulting standard authors, reliable data, and prominent engineers, the writer is unable to find any engineer who is willing to assume the responsibility of allowing an average of less than 30 lbs. per sq. ft. horizontal pressure on the exposed windward side of high buildings."

Probably the most important contribution to the subject of wind-pressure up to the present time is the paper of Mr. Julius Baier before the American Society of Civil Engineers.* From this paper, which gives a most complete discussion of tornadoes, and their causes and effects, the following quotations are taken:

"The St. Louis tornado was but one of a number accompanying a general storm that moved through Missouri and Illinois. As far as known it was not more violent than many others that have been observed. Its great destructiveness was merely incidental to the fact that its path crossed a territory embracing a large and closely built city. It gave evidence that wind pressures existed at least equivalent to or greater than 20 lbs., 60 lbs., and 85 to 90 lbs. per sq. ft. over considerable areas. Whatever the actual distribution may have been, the effects were those of such pressures uniformly distributed over the areas of the respective structures. These pressures were measured by their results in exactly the same manner in which they are ordinarily assumed to act, with the consequent elimination of all uncertainties usually involved in readings of pressure-gauges or deductions from anemometer records, and they are to that extent positive and definite. In addition, there were indications that a pressure of somewhere from 20 to 40 lbs. was quite general over a comparatively wide area in, or adjacent to the path of the storm, and that the pressures at higher altitudes were more severe than those measured.

"In view of these facts it appears to the author rational to assume:

"*First.* That the safety and interests of the community and of the owner of the building require a recognition of a wind

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*** See "Wind Pressure in the St. Louis Tornado, with Special Reference to the Necessity of Wind-bracing for High Buildings," Trans. Am. Soc. C. E., vol. xxxvii.**

pressure of at least 30 lbs. per sq. ft. against the exposed surface of the building, with an additional local provision of 50 lbs. for several stories near the top; and that this amount should be safely taken care of by some positive and definite provision in the construction of the frame.

"*Second.* That the vast interests at stake, the amount of capital invested and the comparatively small additional expense necessary would suggest to the owner the desirability of increasing the provision to 40 lbs. per sq. ft.

"*Third.* That the other uncertain elements of safety due to the ultimate strength of the material, the inertia of the mass, and the bracing effect of walls and partitions, should be recognized only as providing against the uncertain and possible higher pressure of the wind which may occur.

"The chief justification of much that seems bold or questionable in the construction of some high buildings lies in the fact that, as yet, none have failed. If the safety of such great structures is to be determined entirely by the logic of the fitness of the survivor, based on a brief and favorable experience, rather than by a rigid analysis, by tried and accepted principles of engineering design, it may ultimately lead to some very deplorable results."

Methods of Wind-bracing.—It has been previously said that the stability of a building must depend entirely either upon the masonry, that is, the inertia or dead weight of the structure, or upon the steel framework. A free standing masonry wall without bracing of any kind will resist considerable wind pressure on account of its inertia or weight. The greater the weight, the greater the resisting moment, and in this way it may be said that all the materials entering into the building act in some degree to increase, by gravity, the static conditions which must be overcome to allow failure. In buildings of moderate height, with solid masonry construction, adequate resistance to lateral deformation may be secured without the

introduction of steel bracing members; but in veneer-construction buildings of considerable height, in which thin protective walls only are used, and in which the window and court areas are large, and the partitions thin and of little value, the lateral strength of the materials entering into the construction of the building, except the steel frame, cannot be counted upon as of any positive value. While the steel frame is more or less reinforced by the weight and stiffening effects of the other materials, still no definite or even approximate values can be given to such items, except their purely static resistance or weight.

Mr. Julius Baier, in his article on wind pressure in the St. Louis tornado, before mentioned, states as follows regarding the necessity for some efficient system of metallic sway-bracing:

"The effect of an extreme wind pressure on a high office building with curtain walls must depend largely on the extent to which the frame of that building partakes of the nature of the skeleton type or the cage type of construction." . . .

"If, now, the building is of the pure skeleton type, it will have only the elements of stability"—given by—"its weight above the floor in question, and possibly some additional bracing of a more or less uncertain value" . . . "and it will fail, just as the elevator failed; it will topple over and fall to one side or towards one corner on the floor below." . . .

"If the building is of the cage type, it will stand safely under a wind pressure that will destroy the skeleton building. While the failure of the walls at any story may reduce the rigidity somewhat, it cannot affect the strength of a framework designed without placing any dependence on the covering. Such a framework will readily carry the lateral stresses from the upper section to the section below." . . .

"The St. Louis tornado passed within less than a mile of the office buildings in that city. Fortunately it made no test

of the buildings, but it has left some definite evidence of the possible force of the wind and of the action of this force on the materials of construction. While it raises anew the question as to the amount of wind force which should be provided for in designing high buildings, it raises with more emphasis the question as to the method of providing for this force after its amount has been assumed. Any dependence placed on curtain walls and partitions for lateral strength is open to very grave question. The rigidity imparted to a building by the simultaneous action of the total mass of material under ordinary conditions is no indication of the ultimate strength that may be developed at a critical moment, and the very general failure of the walls under extreme wind pressure further destroys any certainty of such assistance as might be otherwise relied upon. The elements of safety against wind force, exclusive of the strength that may come from the walls and partitions where they exist, are the stability due to weight alone, stability due to the strength and stiffness of the frame, and, when the force is a sudden one, the inertia of the mass resisting motion." . . .

"The amount of metal required for an efficient system of wind-bracing is but a small part of the weight of the metal in the entire frame, and the cost of the latter is only about 10 to 20 per cent. of the expenditure for the entire building, exclusive of the site. The cost of the wind-bracing can represent, therefore, only a very small proportion of the total capital invested. When it is considered that any additional metal used to strengthen the cage as a precaution against wind force is equally effective against possible damage due to earthquake shocks or to the unequal settlement of the foundations, and is also an additional margin provided against the weakening effect of corrosion, the slight increase in cost must appear trifling as compared to the amount of the entire investment and the additional protection secured for the property."

"It is somewhat unfortunate that the merits of the design

of the framework are not so readily apparent to the investor, and that this part of the structure is of necessity immediately covered and permanently concealed from view. If the difference in strength and security due to the construction of the frames of some of these great buildings were as generally evident, as, for instance, the difference in strength due to the varying thickness of solid masonry walls was in older forms of construction, there would probably be a more general recognition on the part of the owners of the need of securing the best type of framework."

Full reliance must, therefore, be placed upon some form of lateral bracing in the steel frame. This may be obtained by means of stiffness in the connections, and through the introduction of especial bracing members.

The lateral strength obtained through the various connections of the steel beams, girders, and columns is largely proportional to the details employed in such connections. The difficulty in obtaining proper connections in cast columns, either between themselves, or between the columns and the girders or bracing members, constitutes one of the principal objections to their use. Cast columns will not permit of the use of rivets, nor can any efficient web connections be obtained with connecting girders. The loose bolts destroy the necessary rigidity of the bracing, and in fact the entire stiffness resulting from column and girder connections in steelwork is entirely lacking where cast columns are employed.

No great rigidity can be obtained through the use of steel columns which are joined at each and every floor-level by means of cap-plates. Details are often employed wherein the girders rest upon the cap-plates of the columns, being secured by means of rivets in the lower flanges only of the beams. Such connections are worth little. A better detail is to provide both top and bottom flange connections, using an angle riveted to the column for the top connection, with a leg long enough

and a cap-plate wide enough, to secure four rivets in each flange. A still better detail is to provide special brackets on continuous column shafts, so that the girders may have both top and bottom flange connections, besides web connections directly to the column.

Considerable stiffness may be secured by means of using continuous column splices, as illustrated in Figs. 137 and 138, Chapter VII, where the columns are made in two-story lengths, and staggered as to splices, that is, adjacent columns breaking joints in alternate floors. If this method is employed throughout the building it will add materially to the resultant stiffness, but no very definite value can be placed upon such methods, even providing efficient web connections are made with the girders.

Absolutely positive results can be obtained only through the use of some definite form of metallic bracing. This may be in the form of sway-rods, portals, or deep girders between the columns, a selection depending largely upon circumstances.

Truss-rods, portals, or lattice or plate girders constitute the most definite types of wind-bracing ordinarily employed, and one of these systems should be used where either great strength or positive assurance is desired.

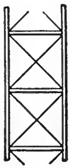

Fig. 143. (1)

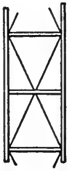

Fig. 144. (2)

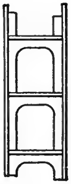

Fig. 145. (3)

Fig. 146. (4)

Fig. 147. (5)

Methods of Wind-bracing.

Bracing by means of sway-rods is shown in Figs. 143 and 144. This system is economical, and easy of erection, the only difficulty lying in the manner in which the sway-rods require

a wall or partition to contain and conceal the members. The locations of doors, etc., may sometimes be arranged to better advantage by making the rods pass through two stories, as in Fig. 144.

Portals, as in Fig. 145, can be used in place of sway-rods where conditions as to corridors, doors, etc., prohibit the crossing of such spaces. In this system the transmission of the strains in the portal members is indirect, and their use is not generally considered economical.

Knee-braces, as in Fig. 146, may only be considered as a partial means of wind-bracing, or as supplementing the stiffness in connections secured elsewhere in the structure. These are not to be recommended as the only means of bracing in any important work, but are used rather as an act of necessity where only partial bracing is required. They can be conveniently arranged in the exterior walls either above or below the girders, or, if required, both above and below, without interfering with the architectural requirements as to windows or other openings. Any form of knee-bracing requires great exactness in manufacture, and care in erection.

Lattice girders, as in type (5), Fig. 147, now constitute a very common form of wind-bracing. In some instances, as in the Reliance Building, Chicago, previously illustrated, plate girders are used instead of latticed members. In this type of bracing the wind stresses are transferred to the ground on what is often called the "table-leg principle," that is, each story is made rigid in itself, the columns being figured as vertical beams to resist the lateral flexure due to the wind forces.

Wind-bracing must reach to some solid connection at the ground. It should also be arranged in some symmetrical relation to the building outlines. If the building is narrow and braced crosswise with one system, the bracing should be midway, while if two systems are employed, they should be placed equidistant from the ends. This symmetry is necessary to

secure the equal services of both systems, thus preventing any twisting tendencies.

Each type must be figured properly, as the strains in the horizontal members and the columns are essentially concerned in the calculations. The problem is not capable of exact solution, owing to several indeterminable factors that enter into the computations, and the consequent equal number of assumptions that must be made. The stresses in the wind-bracing will be maximum when the direction of the wind is normal to the exterior wall, or parallel to the plane of bracing. This condition is, therefore, assumed. A further assumption is made that the floors are sufficiently rigid to transmit the horizontal shears due to wind.

The external forces will be the same whichever of the five methods, shown in the figures above, is used, provided the exposed areas, panels, etc., are the same. The horizontal external force at any panel point will be equal to the distance between the systems (at right angles to the bracing) times the distance between floors half-way above and half-way below, times the assumed wind pressure per square foot. The total shear at any point equals Σ, or the sum of, the forces at or above the point taken.

These shears are undoubtedly reduced to some considerable extent through many practical considerations. The dead-weight of the structure itself, the resistance to lateral strains offered in the stiff riveted connections between the floor systems and the columns, the stiffening effects of partitions (if continuously and strongly built), and linings, coverings, etc., all tend to decrease the distorting effects of the wind pressure. But, in view of the uncertainty in regard to the efficiency of these latter considerations, they may not be relied upon, and are therefore disregarded in the calculations.

Sway-bracing, Analysis of.—The simplest case of wind-bracing is shown in Fig. 143. Considering one bay alone as

braced, the system may be analyzed as follows: Referring to the upper story of a framework, as shown in Fig. 148, $P_1 =$

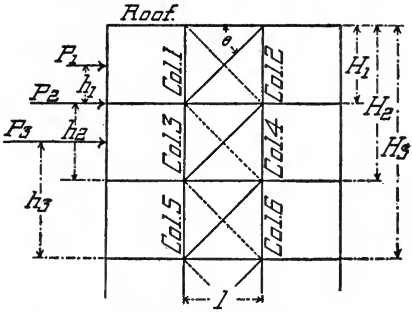

FIG. 148.—Figure showing Analysis of Sway-rod Bracing.

pH_1L_1, where P_1 = resultant wind pressure on upper story, p = unit-pressure, and H_1 and L_1 equal respectively the height and width of the area affecting the bracing in the panel under consideration. $\frac{P_1}{2}$ must then be the horizontal component of the stress in the diagonal, and the tension in this diagonal, making an angle θ with the horizontal, must be

$$T_1 = \frac{P_1}{2} \sec \theta.$$

The diagonal tension in the second story from the top will be $T_2 = \left(\frac{P_1}{2} + P\right) \sec \theta$, where P = wind pressure on any single story, assuming them to be of equal height. $\frac{P_1}{2} + P =$ compressive stress in the horizontal strut at the top-floor level. In like manner, $T_3 = \left(\frac{P_1}{2} + 2P\right) \sec \theta$.

The tension in the diagonal rods will cause a decrease in loads on the windward columns, and an equal increase in loads

on the leeward columns. Calling this increase r decrease V_1, we have

$$V_1 = \frac{P_1h_1}{l}, \text{ where } h_1 = \frac{H_1}{2}.$$

In a similar manner,

$$V_2 = \frac{P_2h_2}{l}, \quad V_3 = \frac{P_3h_3}{l}.$$

V_3 must equal V_2 + the vertical component of the diagonal T_3, or $V_3 = \frac{P_2h_2}{l} + T_3 \sin\theta$. This will serve as a check on the calculations.

These wind loads V_1, V_2, etc., must be added to all the other regular loads on the columns. In the columns 1, 3, etc., the direct or dead-loads carried by the columns resist the upward vertical components of the stresses in the rods connected to the bottoms of these columns. Thus the dead-load in column 3 is reduced by the full amount of the upward compressive strain from wind in that column, or V_2, and if this amount were to equal or exceed the dead-load in column 3, tension would occur in the connection of this column to the one below.

It will be seen that the increment to the stress V at each floor may be eccentric, as shown in Fig. 155, the length of the arm equalling the distance from the point of attachment to the horizontal strut, to the centre of the column itself. If this connection were at the axis of the column, the eccentricity would be reduced to zero, and the eccentric load become a dead-load.

Take the case of a typical skeleton building, fourteen stories in height, of 12 ft. each, 24-ft. front, and columns spaced 12 ft. apart in the depth of the building. Assuming that stiffness against side-yielding alone is necessary, place diagonal members in each story, as in Fig. 149, utilizing the floor-girders as struts, with the columns as chords. At 30 lbs.

per sq. ft. wind pressure the panel-load equals 4,300 lbs. Considering the protection afforded by neighboring buildings, the point of application of the resultant wind pressure will be taken at two-thirds of the height of the structure above ground.

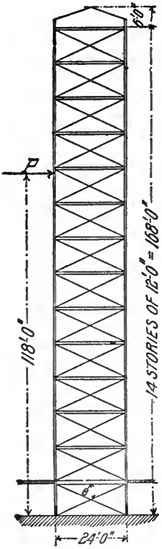

Fig. 149. — Figure showing Typical Sway-rod Bracing.

The total shear will then equal about 60,000 lbs., or 30 tons. In the basement panel, then, sec $\theta = 1.12$, giving 33.6 tons tension in the cellar diagonal. The moment of the resultant wind pressure $= 30 \times 118 = 3,540$ foot-tons, and this, divided by 24, gives $147\frac{1}{2}$ tons compression at the leeward foundation. The vertical component of the basement diagonal $= 15$ tons, leaving a tension of $132\frac{1}{2}$ tons on the windward column.

The dead weight, including iron, walls, floors, filling, etc., will equal about 250 tons for one foundation, while even for a building with no filling or partitions completed, the dead-weight is still some 200 tons, thus rendering anchorage unnecessary.

If, in the same cross-section of the building, n bays not adjacent are braced by means of diagonal rods, the tension T becomes $T_1 = \frac{P_1}{2n} \sec \theta$, and $V_1 = \frac{P_1 h_1}{nl}$.

The bracing in Fig. 144 may easily be analyzed in a manner similar to the above.

Sway-bracing, Examples of.—One of the highest buildings in Chicago is the Masonic Temple, 273 ft. 10 ins. from grade to top of coping. A cross-section of this building is shown in Fig. 150, with one system of bracing-rods. It will be seen that a combination of forms (1) and (2) was used, the

bracing being arranged to suit halls and doorways. In this building the sway-rods were not connected to the floor-beams themselves, but to special I-beams placed between the columns and just below the floor system.

In "The Fair" Building, system (1) was used, but with lattice girders from column to column, serving as struts and floor-beams at the same time. Gusset-plates were dropped below the girder to receive the pins for connection with the turnbuckle rods.

One of the simplest examples of system (1) of wind-bracing was described by Mr. C. T. Purdy in the *Engineering News*, vol. xxvi., No. 52, the example referred to being the Venetian Building, Chicago. The floor plan of this building is shown in the accompanying figure (151), the four

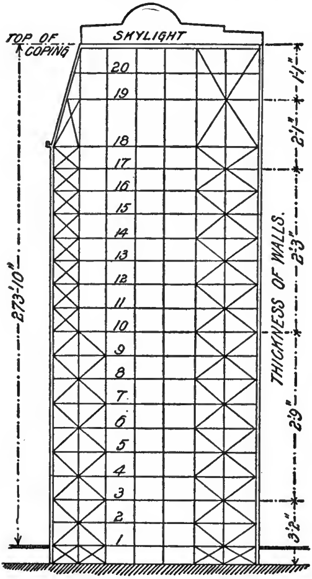

FIG. 150.—Cross-section of Masonic Temple, Chicago, showing Wind-bracing.

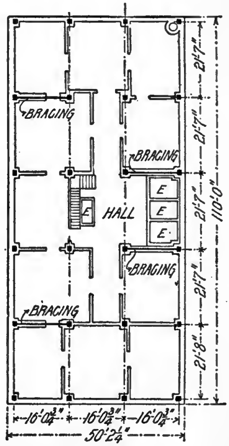

FIG. 151.—Floor Plan of Venetian Building.

sets of sway-rods being located as marked. Each set of bracing is therefore figured to resist a wind pressure for an area the horizontal width of which is equal to one-fifth the depth of the building, and the height of which is the height of the building. The area tributary to each floor × 40 lbs. equals the horizontal shear at each floor or panel-point, while the total shear at any floor equals the sum of the shears acting on the panel-points directly above, as we have seen before. It was not considered necessary, however, to carry the whole amount of this shear into the steel bracing. The practical considerations which tend to diminish the distorting effect due to a lateral force, decided that but 70 per cent. of these shears needed to be cared for by the bracing, leaving 30 per cent. to be taken up by the other factors. The strains and sections for one bay are here given (Fig. 152).

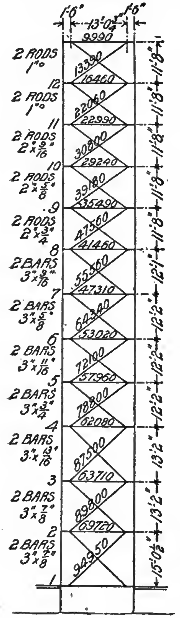

FIG. 152. — Wind-bracing in Venetian Building.

All the columns affected by this bracing were made continuous from the foundations to the second-floor level, and portals were used to take the place of the diagonal rods in two instances where rods were out of the question. This occurred on a main floor devoted to large banking-rooms. The bending moments due to these portals were taken up in the columns. In the case where the rods came down to the first-floor level, the bottom strut was connected to the columns so as to take both tension and compression horizontally, as well as to resist the component of the rod strains. This insured the resistance of both columns to the horizontal thrust of the strut, whichever pair of rods was strained, and

the columns were calculated to resist the bending moment incurred, as well as to carry the regular column-loads.

With the use of the portals, the columns were designed to resist the bending moment which the stopping of the rods necessitated, and as a further assurance that these connections should be as strong as the rest of the system, the top connections of all of the first-floor beams were omitted, and the clearance spaces between all the beams and columns were driven tight with thin metal wedges, until the girders and beams passing along the column axes were continuous and in compression out to the sidewalk walls, which latter are backed by the solid street.

The horizontal channel-struts are shown in Fig. 153.

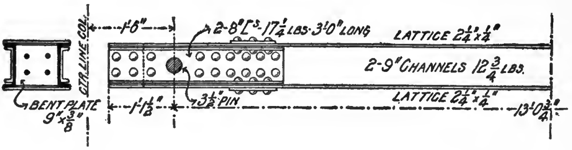

FIG. 153.—Detail of Channel-struts. Venetian Building.

They were used as shown up to and including the seventh floor. A lighter section was used for the floors above. A slight connection only was made between the channel-struts and the columns. The struts were planed at both ends, with no clearance, thus making butt joints with the columns. A bent plate between the channels provided holes for four rivets connecting to the columns, but they were hardly necessary. Underneath the ends of these struts a cast-iron block was bolted to the column and supported by two bracket-angles beneath, with sufficient rivets to resist the vertical compression of the rods in this direction (see Fig. 154).

Above the ends of the struts other cast-iron blocks were used, planed top and bottom, thus allowing them to fit in

tightly between the tops of the struts and the cap-plates of the columns. These blocks, therefore, fitted into the recesses made by the flanges of the Z-bars so closely that the $\frac{3}{4}$-in. cap-plates were brought into direct shear entirely around three sides of the blocks. The shear resistance of the plate, together with the weight of the beam on it, was more than sufficient to resist the upward vertical component of the rods. Such cast-iron blocks in this connection are very convenient

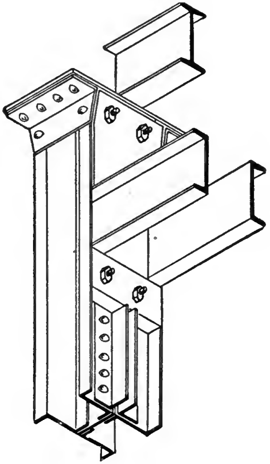

FIG. 154.—Detail of Channel-Strut Connections. Venetian Building.

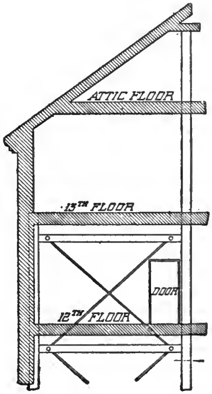

FIG. 155.—Partial Cross-section of Venetian Building.

for use, for it often happens that the bracket-angles cannot be brought directly under the channels of the struts, and the medium between the strut and the bracket-angles must act as a beam as well as a filler. Fig. 155 shows a partial cross-section of the building with doorway, etc. This shows the reason for placing the pin-points so far from the column-centres. The channel-struts are reinforced with cover-channels

to resist the bending moment on the strut caused by thus moving the pin-centres.

The diagonal rods in this building were proportioned on a basis of 20,000 lbs. per sq. in. All rods had turnbuckles, and no rods were of an area less than $\frac{7}{8}$ in. square. The Ashland Block, by Burnham & Root, Chicago, has longer struts than those in the Venetian Building, 15-in. channels being used in the floors, acting both as struts and floor-beams.

Portal Bracing (3), Analysis of.—The third method of wind-bracing, called the portal system, may be analyzed as follows (see Fig. 156): Taking the upper floor first, the

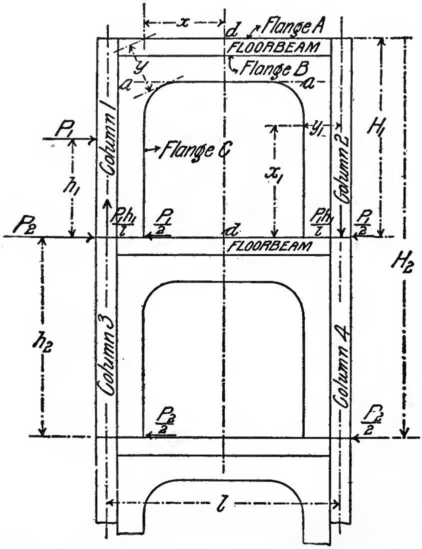

FIG. 156.—Figure showing Analysis of Portal Bracing.

external force P_1 may be considered as producing equal horizontal reactions at the bottoms of the portal legs, or at the

floor-level, equal to $\frac{P_1}{2}$ each. A wind moment M is also produced at this floor-level, or,

$$M = P_1 h_1, \text{ where } h_1 = \frac{H_1}{2}.$$

Owing to the rigidity of the framework, this wind moment will be resisted by the resisting moment of the column sections, and by the portal connections at the floor-line. This resisting moment must equal $\frac{fI}{y_1}$, where $f =$ the unit-strain on extreme fibres, $y_1 =$ distance of extreme fibres from the neutral axis, and $I =$ moment of inertia of the section. But $M = P_1 h_1$, hence $f = \frac{P_1 h_1 y_1}{I}$.

I will be slightly different on the two sides of the neutral axis. On the compression side of the bay, I will be taken as the moment of inertia of the section of the column and the portal, while on the tension side, I must be taken for a section of the column and the bolts securing the portal to the floor-beam or to the portal below. If a splice occurs in the column on the tension side, I must be taken for the sections of the bolts connecting the cap-plates of the column, and for the bolts through the portal and floor-beam.

The decrease of load on the one column, and the equal increase in load on the other column will be as before, or $V_1 = \frac{P_1 h_1}{l}$. In column 2, the vertical column-load V_1 due to wind must be added to the regular column-load, the same as in previous discussion. V_1 must also equal the shear on all vertical planes.

The horizontal shear along the line $aa = P_1$, while the horizontal shear in either leg or portal or at bottom of leg $= \frac{P_1}{2}$. These shears will determine the thickness of the webs.

The connections of the portal to the column on either side must equal the total vertical shear.

Taking moments about the line *dd*, it will be found that $\Sigma M = O$. That is, there is no bending moment along the line *dd*, and neither the floor-beams nor portals are strained by bending moment along this line.

For a maximum stress in the flange C take a point in flange A, distant x from line *dd*, and distant y, at right angles, from flange C. Then x times the vertical shear divided by y = stress at section taken, and this is maximum when $\frac{x}{y}$ has its maximum value. The stress in the flange A may be obtained in a similar manner.

The leg of the portal, including column 2, may also be taken as a cantilever, with the two forces $\frac{P_1}{2}$ and V_1 acting on it. The flange C will be in compression, the column itself acting as a tension chord. Assume a point on the centre line of the column, distant x_1 from bottom of leg, and at distance y_1 from the flange C, at right angles. Then $\frac{P_1 x_1}{2y_1}$ = strain in flange C, and this is maximum when $\frac{x_1}{y_1}$ is maximum. There is a slight error in this treatment, but it is on the side of safety. If the flange C is proportioned for these maximum stresses, the requirements will be fulfilled.

In the second story from the top, $V_2 = \frac{P_2 h_2}{l}$, considering $P_2 = 2P_1$, or that the stories are of equal height. The concentric load V_1 in column 2 from the column above, and its equal reaction, may be omitted in a calculation of the strength of the portal-bracing (as they are applied along the same straight line), as may also the equal negative effects in column 1.

The vertical shear in this second-story bracing will equal $S_2 = V_2 - V_1$. The horizontal shear across the top of the portal $= P_2$, while in either leg the shear $= \frac{P_2}{2}$.

Portal-bracing, Examples of.—One of the first attempts at a portal system in building construction was through the use of a portal-strut used in the older portion of the Monadnock Building, as in Fig. 157.

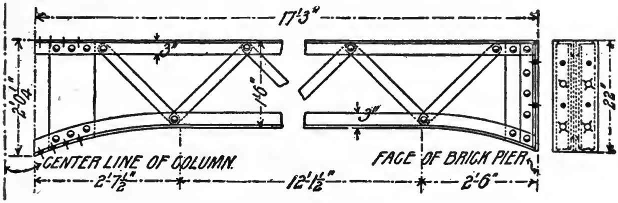

FIG. 157.—Portal-strut used in Monadnock Building.

The portal system (3) was used in the Old Colony Building, Chicago, completed in 1894. The portals are placed at two planes in the building—a cross-section of one set being shown in Fig. 158. Wind pressure was figured at 27 lbs. per sq. ft.

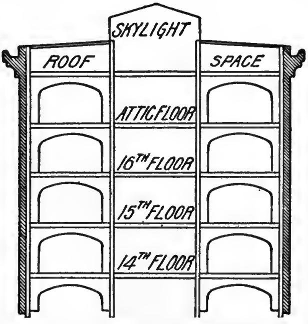

FIG. 158.—Cross-section showing Portals in Old Colony Building.

on one side of the building at a time. Each portal was calculated independently for the sections of both top and bottom

flanges, thickness of web, cross-shear on rivets connecting the curved flanges, and for all splices and connections. A detail of one portal is shown in Fig. 159. This arrangement of

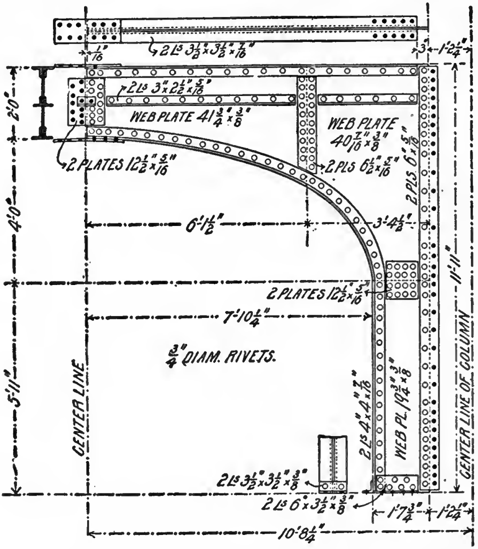

FIG. 159.—Detail of Portal in Old Colony Building.

wind-bracing proved very satisfactory in all respects, and, according to the designer, was cheaper in the end than the sway-rods provided in the first design; but the writer would question whether portal-bracing can be provided cheaper than tension-rods, as claimed. With good details in connections and proper regard for their location in the original planning of the building, sway-rods can be used without great expense or trouble. The portal arrangement certainly makes a fine interior appearance if the arched openings are given a slight decorative treatment in plaster, as was done in the Old Colony Building. The floor plan will generally govern the use of

either one or the other system, whether the rooms are to be connected by large openings or small doorways.

Knee-braces (4), Analysis of.—The system of knee-braces, or arrangement (4) for wind-bracing, is not an economical method, as it produces heavy bending moments in both the horizontal struts and in the columns themselves. This system may be analyzed as follows (see Fig. 160):

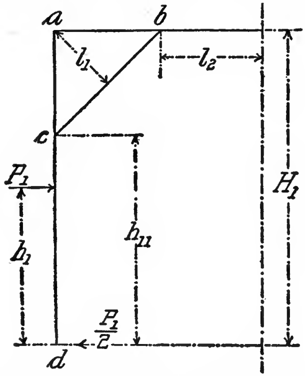

FIG. 160.—Figure showing Analysis of Knee-bracing.

The shear at the top-floor level will be $\frac{P_1}{2}$ at each column.

Then as before, $V_1 = \frac{P_1 h_1}{l}$.

The tension in the brace *cb* is nearly

$$T_1 = \frac{P_1}{2} \cdot H_1 \cdot \frac{1}{l_1} = \frac{P_1 H_1}{2 l_1}.$$

There will be an equal amount of *compression* in the opposite brace. This suggests the use of knee-braces capable of resisting both compression and tension. There will be a bending moment at C whose value is approximately $M = \frac{P_1}{2} \cdot \frac{h_{11}}{2} = \frac{P_1 h_{11}}{4}$. The factor $\frac{h_{11}}{2}$ is used, as the column is considered as

square-ended and fixed by the static load and by bolts. This bending moment will also exist at d.

At b there will be a bending moment $M_1 = V_1 l_2 = \frac{P_1 h_1 l_2}{l}$.

Knee-braces, Examples of.—This type of wind-bracing was used in the Isabella Building, by W. L. B. Jenney, architect, as shown in Fig. 161.

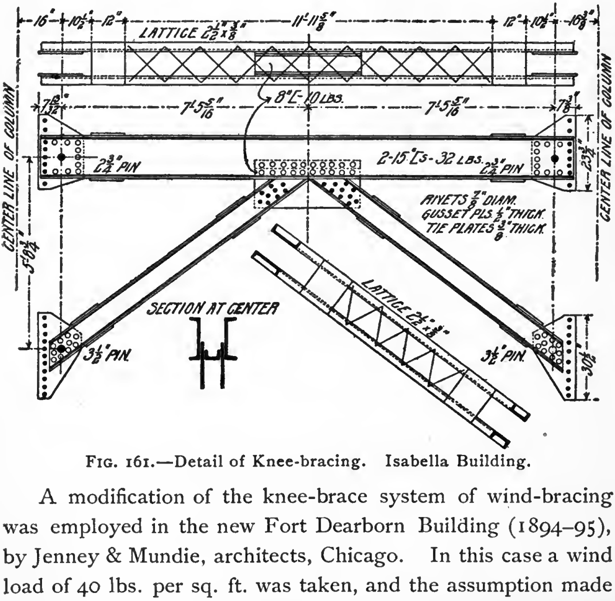

FIG. 161.—Detail of Knee-bracing. Isabella Building.

A modification of the knee-brace system of wind-bracing was employed in the new Fort Dearborn Building (1894–95), by Jenney & Mundie, architects, Chicago. In this case a wind load of 40 lbs. per sq. ft. was taken, and the assumption made that 25 per cent. of this wind load would be resisted by the rigid connections provided between the columns and the floor system, leaving 75 per cent., or 30 lbs. per sq. ft., to be taken up by the exterior columns. This was done by using channel girders between the columns in the exterior walls, with gusset-plate connections to the columns, as shown in Fig. 162, 10-in. and 12-in. channels being used generally. In the lower stories,

where the wind moment necessitated it, a double system of gusset connections was used, under and above the channel girders.

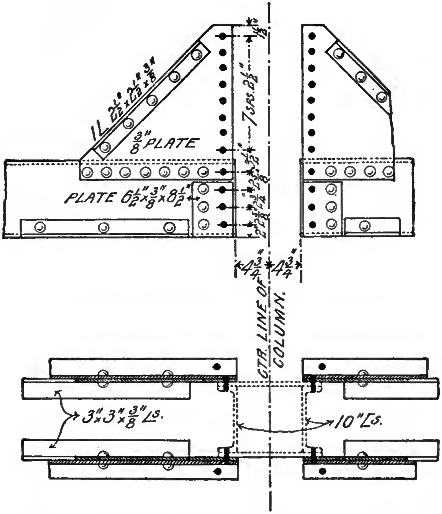

FIG. 162.—Detail of Channel-struts and Gussets.
Fort Dearborn Building.

Lattice Girders (5), Analysis of.—Referring to Fig. 163, the external wind pressure for the panel in question may, as before, be represented by P_n, this being for a superficial area extending half way to the next bracing members, both horizontally and vertically. $\frac{P_n}{2}$ may then be considered as applied in a line with each chord of the girder. The horizontal shear due to the force P_n must then be resisted by the two columns at any and all points between the lower line of the girder and the top line of the girder below. Hence the foot of each column must resist the shear $\frac{P_n}{2}$. Also, if P_s represents the

shear from all the stories above the bracing in question, $\frac{P_s}{2}$ will equal the shear at the foot of each of the upper columns, and $\frac{P_n + P_s}{2}$ will be the total shear at the foot of each of the columns in the story under consideration.

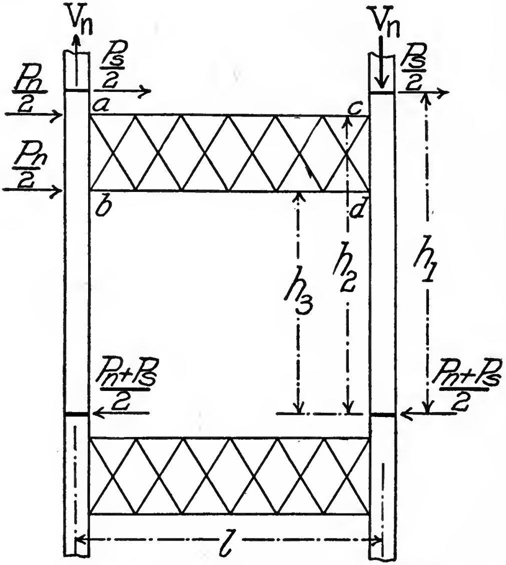

FIG. 163.—Figure showing Analysis of Lattice-girder Bracing.

If the compression in the leeward column or the decrease in load in the windward column is called V as formerly, for the story in question, and if V_n represents the same forces in the story above, then

$$V = V_n + \frac{P_s h_1 + \frac{1}{2}P_n(h_2 + h_3)}{l}.$$

This value of V is a live load due to the wind forces, and must be added to the other column loads as in previous examples.

The compressive stress in the upper flange of the girder, or member ac,

$$= \frac{\frac{1}{2}P_s(h_1 - h_3) + \frac{1}{2}(P_n + P_s)h_3}{h_2 - h_3} + \frac{1}{2}P_n.$$

The stress in the lower flange of the girder, or member bd, which is also compression,

$$= \frac{\frac{1}{2}P_s(h_1 - h_2) + \frac{1}{2}(P_n + P_s)h_2}{h_2 - h_3} - \frac{1}{2}P_n.$$

Considering the columns as fixed at both ends, the maximum bending moments will be at the points b and d, and will be equal to

$$\frac{1}{2} \times \frac{P_n + P_s}{2} \times h_3 = \frac{P_n + P_s}{4} h_3.$$

The columns must be designed to resist this bending moment as well as the vertical loads. This would suggest that in narrow buildings of considerable height, the columns be made of such form as to give a greater width or depth in the narrow direction of the structure than is provided lengthwise, thus providing in the form for this additional moment.

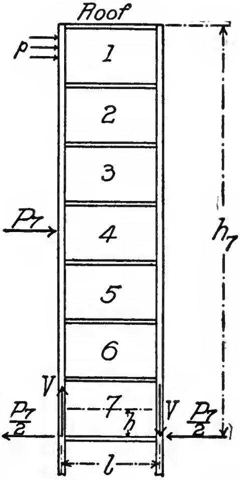

FIG. 164.—Analysis of Lattice-girder Bracing.

The values V previously derived may be obtained somewhat more simply by using the notation given in Fig. 164.

Let p = wind pressure per lineal foot of height;

h_7 = distance from roof to foot of columns of story in question;

h = distance from foot of columns in question to line midway between girders in story in question and story below.

Then $P_7 = p(h_7 - h)$, and

$$V = \frac{P_7\left(\frac{h_7 - h}{2} + h\right)}{l} = \frac{P_7(h_7 + h)}{2l}.$$

This value of V should correspond with that previously given.

Lattice Girders, Examples of.—Lattice girders as here described are almost invariably used in supporting the floor-loads for the tributary areas, the floor-beams being carried at the panel-points of the latticed members. Also, when located within the exterior walls, the girders serve as spandrel supports as well, thus carrying the wall-loads story by story. The girders consequently perform a twofold service—they support vertical floor- and wall-loads and, at the same time, serve as struts for the transmission of wind strains.

In proportioning such members, they should first be calculated for the vertical loads—either for the floor- or wall-loads, or both, after which the sections of the upper and lower flanges should be increased to provide for the additional wind strains here given. The diagonals or lattice members should also be somewhat increased in section, thus providing for sufficient rigidity between the flanges. The girders are usually made the full depth of the spandrel, reaching from just above the top of one window to immediately below the sills of the windows in the next story above.

In some cases, where double-story column lengths are employed, lattice girders of this type are placed at the column joints only, that is, in alternate stories. In the intermediate stories, the usual spandrel-beams or channels are inserted. This practice greatly increases the bending moments on the columns, and is not advisable in extremely high or important work.

Fig. 165 illustrates a diagram elevation of the framework of the south wall of the Park Row Building, showing the

combined use of lattice, plate, and box girders, angle-braces, and sway-rods. A good example of lattice girders was also employed in the Tract Society Building, New York City.

In the Reliance Building, Chicago, 55 ft. wide and 200 ft.

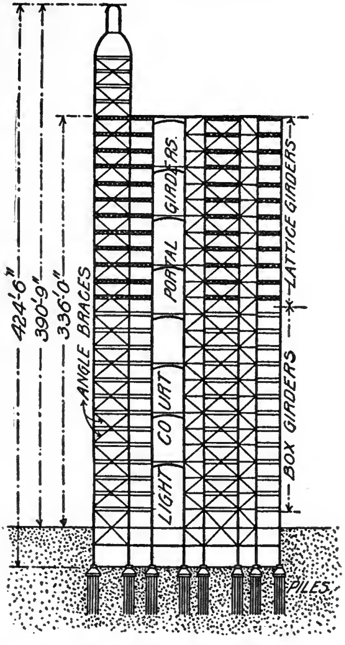

FIG. 165.—Diagram Elevation of Park Row Building, showing Wind Bracing.

high, 24-in. plate girders were used between all outside columns, the connections to the columns being made vertically to the webs of the girders, as shown in Figs. 103 and 126.

In the very narrow 10-story Worthington Building, Boston, the wind strains were cared for without the aid of any diagonals or portals by making the corner columns of two heavy web plates, one in each wall plane, which thus acted as vertical

plate girders. The intermediate wall columns are also built up of plates and angles, but of large superficial areas. The columns are united by continuous lines of plate girders, serving as both wall- and floor-girders as well as for the transmission of wind strains, the total wall area thus appearing much like a solid metal diaphragm or sheet, perforated by window areas.

In this building some of the plate girders are arranged with sliding-shelf seats and slotted holes, so as to provide for expansion and contraction under variations of temperature.

Deflection or Vibration.—For the theoretical limit in the height of a building, considering the wind pressure, we may assume that the wind acts against the building in a horizontal direction, so that the structure may be taken as being under the same conditions as a uniformly loaded beam, fixed at one end and with the other end free. If this were actually the case with a steel beam, we should make the depth of the beam such that it would deflect less than the amount necessary to crack the plaster. If the beam were supported at both ends, this depth would be one twentieth of the span.

The lengths under these two conditions, to secure the same deflections, must bear the relation one to the other as 0.57 to 1.

If, then, we have an office building or any skeleton structure 25 ft. wide, and make the height twenty times the width, the building would be 500 ft. high, and reducing this in the above ratio, we have 285 ft.

This height would give a theoretical deflection of some 8 ins. or 9 ins., which would throw the centre of gravity of the upper wall beyond the outer edge. The maximum allowable deflection would be about 2½ ins. or 3 ins., and this would give a height of from 70 to 95 ft.

The load effect on a uniformly loaded cantilever is four times that for a uniformly loaded beam supported at both ends. If we work on the assumption that the building is analogous

to the cantilever beam, and make its height one fourth as great as we would if it were supported at both ends, we should have the depth to the length about as 1 to 5. This would give a height of 125 ft.

Some careful experiments, however (see *Engineering News*, March 3, 1894), on the deflections of tall skeleton-construction buildings in Chicago, tend to show that any actual deflections in well designed and carefully constructed buildings, under very heavy winds, are far less than any theoretical assumptions. Two sets of tests were made, one on the Monadnock Building of seventeen stories, and the other on the Pontiac Building of fourteen stories. Observations were made with transits set in sheltered positions, and these observations were checked by means of plumb-bobs, suspended in the stair-wells from the top floor.

The vibrations in the Monadnock Building from west to east, or in its narrow direction, were from $\frac{1}{4}$ in. to $\frac{1}{2}$ in. The plumb-bob test, however, showed the greatest variation to be in a north and south direction, or longitudinally; but as the walls in three of the four separate divisions of this building are of solid brickwork, from 3 ft. to 6 ft. in thickness, and the length is several times the breadth, it is difficult to believe that any actual longitudinal deflection could be detected.

In the transverse deflections the transits showed a greater deflection in the veneer portion of the building than in the more solid parts, as would very naturally be expected. The time of a complete vibration was two seconds.

The experiments on the Pontiac Building, which is of the veneer type, compared very closely with those on the Monadnock Building, except that the amplitude of the vibration was less in the former building, due to its somewhat more sheltered position. The same peculiarity of an apparently greater longitudinal vibration was noticed here also. The wind was from the northwest, and registered eighty miles per hour.

Veneer or skeleton construction has been adopted in San Francisco, where the fear of earthquakes has, heretofore, been sufficient to keep investors from erecting high buildings. The new Chronicle Building and the Croker and Mills buildings are of the veneer type, twelve stories and over in height, and have served as precedents in that locality.

In 1897, a still higher structure, namely, the Spreckels Building, was erected after advanced methods. This building is for office purposes, the height being 19 stories, or 300 ft. above the sidewalk. An earthquake shock which disturbed that locality in the spring of 1898 was reported to have caused the building to rock and sway, but to have left it practically uninjured.

Building Laws.—The building laws of Greater New York require the following provisions as regards wind forces:

"All structures exposed to wind shall be designed to resist a horizontal wind pressure of thirty pounds for every square foot of surface thus exposed, from the ground to the top of the same, including roof, in any direction.

"In no case shall the overturning moment due to wind pressure exceed seventy-five per centum of the moment of stability of the structure.

"In all structures exposed to wind, if the resisting moments of the ordinary materials of construction, such as masonry, partitions, floors, and connections are not sufficient to resist the moment of distortion due to wind pressure, taken in any direction on any part of the structure, additional bracing shall be introduced sufficient to make up the difference in the moments.

"In calculations for wind-bracing, the working stresses set forth in this Code may be increased by fifty per centum.

"In buildings under one hundred feet in height, provided the height does not exceed four times the average width of the base, the wind pressure may be disregarded."

The Chicago building ordinance makes the following requirements:

"In the case of all buildings, the height of which is more than one and one-half times their least horizontal dimension, allowances shall be made for wind pressure which shall not be figured at less than thirty pounds for each square foot of exposed surface. In buildings of skeleton construction the metal frame must be designed to resist this wind pressure."

The building laws of Boston and Philadelphia contain no reference to wind pressures.

CHAPTER IX.

FOUNDATIONS.

No part of the architect's or engineer's work requires more care than the successful planning and carrying out of the foundation design. The importance of an adequate foundation has, fortunately, been pretty generally realized at all times; but where the architect or engineer was formerly called upon to meet only comparatively simple conditions in building practice, namely, the designing of offset masonry foundations for buildings of no great height where little or no thought for adjacent work was required, present conditions of foundation design in large cities often make an exceedingly complex problem. The architect must now deal with large concentrated loads for buildings of great height and often of very small area; he must frequently build on treacherous soil, and thus be required to find expedients to supplement the natural weakness of the foundation bed by artificial means; the safety of surrounding structures must be preserved during building operations; and means must be provided to guard against the undue settlement of, and consequent damage to adjoining buildings.

Foundation design for important structures will be found to differ widely in various cities or localities, owing to the great differences in the character of the underlying material. Thus, in Chicago, surface foundations predominate in high building design; in Boston, piles are used very extensively; and in New York City pneumatic foundations to bed-rock or hard pan have been largely used since the introduction of skeleton methods. All of these types will be explained more or less fully in this chapter, but successful and economical

foundations are so largely matters of good judgment and experience, that general descriptions only may be attempted as guides to conditions arising in actual practice.

Bearing-power of Foundation Materials.—The safe loads which may be applied to foundation soils naturally vary greatly according to the character or composition of the stratum to be built upon, or upon the character of the underlying but invisible subs oil. It will not be sufficient to base any decided opinions upon what may be seen only. An examination below the surface is indispensable for all materials except firm rock, unless precedent has unquestionably established safe unit-loads.

Foundation materials vary in reliability from rock bottom, hard and compact and in natural bed, to poorer or "rotten" rock formations, clayey soils, gravel, or, finally, to such unstable bottoms as mud, marshy ground, or quicksand. For complete data as to the bearing power of these different materials, and for a great range of valuable information pertaining to foundations, reference may be made to "A Treatise on Masonry Construction," by Prof. I. O. Baker, or to "A Practical Treatise on Foundations," by W. M. Patton. The following table, giving the average safe bearing-powers of soil, is taken from Prof. Baker's work:

Kind of Material.	Safe Bearing-power in Tons per Sq. Ft.	
	Min.	Max.
Rock—the hardest—in thick layers, in native bed	200	....
" equal to best ashlar masonry	25	30
" " " " brick "	15	20
" " " poor " "	5	10
Clay, in thick beds, always dry	4	6
" " " " moderately dry	2	4
" soft	1	2
Gravel and coarse sand, well cemented	8	10
Sand, compact and well cemented	4	6
" clean, dry	2	4
Quicksand, alluvial soils, etc.	0.5	1

For ordinary soils it is therefore generally safe to assume a capacity of from 2 to 4 tons, or 4,000 to 8,000 lbs. per square foot, while for soft or treacherous soils, or those resting on soft strata, the load should not exceed 1 to 2 tons, or 2,000 to 4,000 lbs.

In building a structure of any importance upon soft or yielding material, either because of the difficulty or expense in reaching a firm bottom, it is not always sufficient that the weight upon the soil should cause no injurious settlement; for if such material as mud or fine wet sand is heavily loaded and *not confined*, the lateral escape of the semi-fluid mass may be permitted by near-by excavations or building operations, or even by excavations at a considerable distance. Such lateral escapement may result in serious settlement to the structure, or in great expense and trouble to adjacent owners, even where the building laws may have been technically complied with. If any possibility of lateral relief exists, equity to all should dictate the use of deep foundations or piles to solid material. It has been claimed that an excess of settlement has resulted in certain structures in New York City, lying near the water-front, due to this flow of underlying soft soil.*

In compact clayey soils, the large experience gained on such foundation material in Chicago goes to show that no perceptible lateral movement occurs; for with very heavy buildings on either side of the street, the soil would naturally follow the line of least resistance, and show upheaval or disturbance of piping and pavements. This tendency has never been noticed, and it is therefore presumed that the settlement which does occur results from the gradual squeezing out of the water in the clay.

Rock foundation is seldom loaded to the full capacity, even under greatly concentrated loads. In New York City, the

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* See "Concerning Foundations for Heavy Buildings in New York City," by Chas. Sooysmith, Trans. Am. Soc. C. E., vol. xxxv.

hard stratum, where not rock, is usually found to be a very firm and compact mixture of silt, clay, and gravel, containing stones of various sizes. This is generally called hard-pan, but is sometimes termed rock on account of its exceeding hardness. The safe bearing capacity is considerably in excess of the usual pressure per square foot for concrete bases, viz., 150 lbs. per sq. in., or 10.8 tons per sq. ft.

Bearing Pressures: Building Laws.—The "Bearing Capacity of Soil" is thus specified in the Greater New York Building Code:

"Where no test of the sustaining power of the soil is made, different soils, excluding mud, at the bottom of the footings, shall be deemed to safely sustain the following loads to the superficial foot, namely:

"Soft clay, one ton per square foot;

"Ordinary clay and sand together, in layers, wet and springy, two tons per square foot;

"Loam, clay, or fine sand, firm and dry, three tons per square foot;

"Very firm, coarse sand, stiff gravel, or hard clay, four tons per square foot, or as otherwise determined by the Commissioner of Buildings having jurisdiction."

The Chicago Building Ordinance requires the following:

"If foundations of other materials than piles are used, they shall be so proportioned that the loads upon the soil shall not exceed the limits for different kinds of soil than those hereafter given, to wit:

"If the soil is a layer of pure clay at least fifteen feet thick, without admixture of any foreign substance excepting gravel, it shall not be loaded more than at the rate of 3,500 pounds per square foot. If the soil is a layer of pure clay at least fifteen feet thick and is dry and thoroughly compressed, it may be loaded not to exceed 4,500 pounds per square foot.

"If the soil is a layer of dry sand fifteen feet or more in

thickness, and without admixture of clay, loam, or other foreign substance, it shall not be loaded more than at the rate of 4,000 pounds per square foot.

"Foundations shall not be laid on filled or made ground, or on loam, or on any soil containing admixture of organic matter.

"If the soil is a mixture of clay and sand, it shall not be loaded more than at the rate of 3,000 pounds per square foot."

The Boston Building Law leaves the determination of the bearing-power of soils to the discretion of the building authorities.

Examples of Foundation Pressures.—The following data will serve to show the actual unit pressures on the soil induced by a number of well-known buildings. These are, however, of little value in determining the allowable pressure for other structures, even though very near to the sites mentioned, as full records of test borings, or samples of the actual materials encountered are required in all cases for a proper determination of bearing values.

As examples of bearing on rock or hard-pan in New York City, at the base of caisson foundations, the Manhattan Life and the Gillender buildings may be cited. The Manhattan Life Building is seventeen stories high, and is supported on 15 caissons, the pressure per square foot at base of caissons being calculated at 10.8 tons per sq. ft. The Gillender Building, supported on caissons sunk to bed rock, causes an estimated unit pressure of 12 tons per sq. ft.

For bearing on sand, the New York "World" Building resulted in a load of 4.7 tons per sq. ft., some of the resultant loads being considerably eccentric. The foundations consist of inverted arches built upon continuous concrete footings, thus resulting in broad belts of bearing areas. The material of the site was fine dense sand.

The St. Paul Building (see Frontispiece) is built upon an

extremely compact sand, overlaid with fine sand. The foundations consist of a steel and concrete grillage covering the entire lot area, the resultant pressure being 3.2 tons per sq. ft. The Spreckels Building, San Francisco, 310 ft. high, is built upon a very similar grillage covering the entire foundation area, the pressure being 4,500 lbs. per sq. ft. on a dense wet sand.

"The Washington Monument, Washington, D. C., rests upon a bed of *very* fine sand 2 ft. thick underlying a bed of gravel and bowlders; the ordinary pressure on certain parts of the foundation is not far from 11 tons per sq. ft., which the wind may increase to nearly 14 tons per sq. ft." *

In Chicago, the soil underlying the city consists of loam or made ground to a depth of 12 or 14 ft. below the sidewalk grade, below which there is a layer of blue clay, sometimes termed hard-pan, from 6 to 10 ft. thick. Below the firm layer, the material changes to different grades of soft and saturated clay, which again becomes hard and firm at a depth of 50 to 60 ft. Limestone bed-rock is found at from 40 to 80 ft. below the street-level.

The upper stratum of hard clay is used for the support of the grillage foundations, and custom has established a unit pressure of from 3,000 to 4,000 lbs. per sq. ft. From 3,000 to 3,500 lbs. has been found to give the best results. "The Fair" Building was loaded to 2,850 lbs. per sq. ft. on the soil, this being more conservative than average practice. The Y. M. C. A. Building loaded the clay to 3,500 lbs. per sq. ft., and the Monadnock Building to 3,750 lbs. per sq. ft.

"In the case of the Congressional Library, the ultimate supporting power of 'yellow clay mixed with sand' was 13½ tons per sq. ft.; and the safe load was assumed to be 2½ tons per sq. ft." †

* See "A Treatise on Masonry Construction," I. O. Baker, page 192.

† *Ibid.*

In Boston, dry compact clay is loaded to 3 tons per sq. ft.

Test Loads.—If foundations are to be constructed in or upon compressible soil, tests of the bearing capacity of the material are desirable if any doubt exists as to safe unit-loads. Such tests are often resorted to where raft or grillage foundations are employed, or for pile foundations.

Tests to determine the bearing-power of the soil at the site of the Chicago Masonic Temple were made by supporting an iron tank on a plate of 2 sq. ft. area.* In one test the plate rested directly on the hard-pan, and in the second test it was placed at the bottom of a hole 2 ft. 4 ins. deep in the hard pan. The tank was gradually filled with water, and the settlements were noted under the varying loads. The time of observations extended over four and six days, respectively, in the two tests. These tests showed that it is safer never to descend below the top of the hard pan in such clayey foundation material as exists in Chicago.

Test loads to determine the bearing capacity of piles are sometimes made by loading a pile or a group of piles with a box of sand or other material, and noting the settlements. Groups of piles were thus tested on the sites of the World's Fair Buildings at Chicago, the piles being driven to different depths, to ascertain the differences in settlement under a uniform load.

The Chicago Library foundations are among the most carefully executed pile foundations in Chicago. Under the walls of this building three rows of piles were driven, and the tests were made as follows: To give the conditions as they would be in the final structure, three rows of piles were driven in a trench, and the middle row was cut off below the other two, thus bringing all the bearing on four piles only (two in each outside row), but thereby allowing the outside rows to derive

* See E. C. Shankland in Minutes of the Proceedings of the Institution of Civil Engineers, vol. cxxviii.

the benefit of the compression of the earth due to the driving of the central row. The work was done by a Nasmyth hammer, weighing 4,500 lbs., falling 42 ins., and having a velocity of 54 blows per minute. The last 20 ft. were driven with an oak follower. The piles were driven at $2\frac{1}{2}$ ft. centres to a depth of 52 ft., 27 ft. into soft clay, 23 ft. into hard clay, and 2 ft. into the hard-pan. Their average diameter was 13 ins., and the area at the small end 80 sq. ins.

The bearing-power of the hard-pan was taken at 200 lbs. per sq. in. Rankine's formula gives about 170 lbs. The extreme average frictional resistance per square inch of the sides of the piles, deduced from experiments under analogous conditions, was 15 lbs. per sq. in. The extreme resistance at the pile point was 200 lbs. $\times$ 80 = 1600 lbs. The average external surface of one pile equalled $(52 \times 12 \times 41) = 25{,}000$ sq. ins. At 15 lbs. per sq. in. this gives 375,000 lbs., or $195\frac{1}{2}$ tons. Disregarding the point resistance, the bearing-power of a pile would be about 187 tons.

Assuming the ultimate crushing strength of wet Norway pine not over 1,600 lbs. per sq. in., and with a factor of safety of 3, the safe load will be not over 533 lbs. per sq. in. The piles were taken at an average area of 113 sq. ins., which gives not over 60,230 lbs. per pile, or about 30 tons. This gives a factor of 3 for crushing, and a factor of 6 for the frictional resistance of the soil. If the timber were loaded at one half its ultimate strength, 45 tons could be used per pile.

A platform to hold a load of pig-iron was built resting on the outside rows of piles, and the weight was gradually increased until at the end of eleven days the mass was 38 ft. high, weighing 404,800 lbs. on 4 piles, or about $50\frac{7}{10}$ tons per pile. Levels were taken at intervals of two weeks, and as no settlement was observed, 30 tons per pile was considered a safe load.

Tests were also made of drawing piles at this site, and an

ordinary pile, driven in clay to a depth of 45 ft., gave 45,000 lbs. resistance.

A very interesting test of the bearing capacity of a foundation soil was made at the time of erecting the St. Paul Building, New York. This structure is 25 stories high, and the ratio of height to width is unusually great, as may be seen in the Frontispiece showing a view of Post-office Square, with the St. Paul Building to the right.

The character of the foundation material was found to consist of bed-rock (at a distance of about 86 ft. below the street-level), overlaid with a fine but extremely compact sand which was considered capable of sustaining at least 4 or 5 tons per sq. ft. The architect, Mr. Geo. B. Post, therefore decided to excavate to the fine sand found just below the water-level, and to cover the entire site with a solid protective layer of concrete, 12 ins. thick, upon which the grillage footings were to be placed. These steel grillages were designed to distribute a uniform pressure of 3.2 tons per sq. ft., with an attendant uniform settlement of $\frac{5}{8}$ of an inch; and as pumping tests had failed to show any disturbances in the adjacent sand, and furthermore, as both the "Times" and "World" buildings had been founded upon practically the same strata of sand nearby, with heavy loading and satisfactory results, it was thought that the proposed construction would prove very satisfactory.

The above decision of the architect, however, was publicly questioned, and as even very slight inequalities of settlement might prove serious in so high and narrow a building, it was decided to make a careful experimental test. This was conducted by Mr. Theodore Cooper as follows:

On the sand bottom of a hole cut in the concrete, a 12-in. by 12-in. stick was placed on end on March 26, 1896, and this was loaded gradually until, on April 8, the gauge showed a settlement of $\frac{19}{32}$ of an inch, under a load of 13,000 lbs. No

additional settlement was caused by pouring water into the test-hole. It was then decided to examine the effects of cutting a second nearby hole in the concrete bed, so a new hole was made 4 ft. 6 ins. from the first. As no new evidence of settlement occurred under the new conditions, 21 ins. of water was poured into the first test-hole, and this was soon visible in the second opening through the effects of moisture in the sand. Both holes were then filled with water and allowed to remain until the following day, and as no added settlement resulted to the test-load, nor any uplifting of sand in the second hole occurred, the test was considered as warranting the architect's design in all particulars.

Test Borings.—Unless repeated precedents of identical conditions exist, an accurate knowledge of the underlying foundation material is plainly a requisite of the utmost importance before an intelligent foundation design can be even approximated. A sufficient number of borings or soundings from which to judge existing conditions will always prove both time and money well invested.

The number of test borings required for any particular site will largely depend upon the nature of the subsoil, and upon the character of the proposed foundations. If the underlying material is known by previous experience to be comparatively homogeneous, and if the character of the foundations is such that reasonable variations in the subsoil are no particular obstacles, a few borings only may be sufficient to give a comparatively accurate knowledge; but if pneumatic foundations are to be employed, or if the character of the substrata is liable to considerable variation as to depth or composition, then it will be found best to provide borings at more frequent intervals,—sometimes several borings within the limits of each pier. For the new Post-office and Government Building in Chicago, only four borings were made, one at each corner of the site, and as these were found essentially alike they were

furnished to the contractors bidding on the foundation contract "as general and not specific information, the contractor assuming all chances as to the formation of the soil."

Test borings may be made in a comparatively simple, inexpensive, and still trustworthy manner as follows:

A section of $1\frac{1}{2}$-in. or 2-in. iron pipe is first driven into the ground as far as possible. A length of $\frac{3}{4}$-in. pipe is then provided with a wedge-shaped end or cutting edge, about 12 ins. long, this being attached by means of an ordinary threaded coupling. Small holes are provided in the faces of the wedge, and the section of smaller pipe, with its wedge end, is then inserted within the large pipe already driven. The upper end of the $\frac{3}{4}$-in. pipe is provided with a special handle or with a hammer end, this being usually about 12 ins. long with a solid handle-bar at right angles to the line of pipe, provided hand pressure is to be used, or with a buffer end or cap in case a ram or weight is employed. In either case, connection is made for water-supply by means of a short elbow which will connect the water-service with the inside of the $\frac{3}{4}$-in. pipe, the water being delivered at a pressure varying from 50 to 100 lbs., depending upon the service. This is sometimes obtained from city pressure, sometimes from a hand force-pump, or even from a steam pump, if upon the premises.

Upon starting the water-supply, the water passes down the $\frac{3}{4}$-in. pipe, through the small holes in the wedge end, and then upwards between the two pipes, bringing the bottom material with it, in suspension, and discharging over the top of the outer tube. As the water scours out at the bottom, the small tube may be gradually lowered into the subsoil either by constantly turning the handle at the top, or by means of a light iron ram, sliding in upright guides and operated by a windlass. In silt or clay it will not generally be found necessary to lower the outside pipe with the $\frac{3}{4}$-in. pipe, as the hole will remain

sufficiently large under the water action alone. In gravel or sand the outer pipe should generally follow the inner one.

To obtain samples of the material being penetrated, the inner pipe is lifted out at intervals of several feet, the wedge end is removed, and a special iron or brass tube is attached, this being usually about 12 ins. long, and slightly contracted at the lower end. This is then lowered to the bottom and pressed for its full depth into the material. The pipe is then raised, and the sample is pressed from the tube and placed in bottles or jars for later examination.

Bowlders or bed-rock can be told by the sound or rebound of the pipe. If bowlders are encountered, a new boring must be started at some distance from the first position. Bowlders, or a thin layer of rock underlaid by a stratum of soft and unreliable character, may easily be mistaken for bed-rock.

Adjoining or Party Walls.—Where modern buildings of considerable height are built next to older structures, the foundations of the new building are almost invariably placed at a lower level than the foundations of the old adjoining building. This is because of the present necessity for sub-basements, in which to place the mechanical plant of the modern building, and also on account of the desirability of carrying the foundations for tall and important structures below the surface-soil, or to hard-pan or solid rock. Party walls, also, where utilized by the newer building, are often required to be extended downwards, to provide deeper basement and sub-basement room in the new structure than exists in the old.

The present Building Code of Greater New York provides that "Whenever an excavation of either earth or rock for building or other purposes shall be intended to be, or shall be, carried to the depth of more than ten feet below the curb, the person or persons causing such excavation to be made shall at all times, from the commencement until the completion thereof, if afforded the necessary license to enter upon the

adjoining land and not otherwise, at his or their own expense preserve any adjoining or contiguous wall or walls, structure or structures from injury, and support the same by proper foundations, so that the said wall or walls, structure or structures, shall be and remain practically as safe as before such excavation was commenced, whether the said adjoining or contiguous wall or walls, structure or structures, are down more or less than ten feet below the curb.''

If license to occupy the premises of the adjoining basement space is not accorded, then the owner refusing to grant such license is obliged to make his own walls secure by proper foundations; but as few owners would deny such a privilege when the cost of protecting or renewing their foundations would fall upon themselves, the responsibility for adjoining or party walls is usually upon the owners of the new structure.

The construction of the older existing buildings is liable to be of a far inferior quality, the foundations often consisting of rubble or dimension stone carried down a short distance only below the basement grade, while the walls which, in the older construction, almost invariably support floor- and roof-loads, are apt to prove of indifferent quality and dangerous to alter in any way. Great care is therefore necessary in the protection of the existing work, and as the driving of new pile foundations close to the old wall or foundation would induce jar or settlement, and as excavation would undermine the support, some temporary method of securing the original walls must be resorted to, while the foundations are being either reinforced or rebuilt. This must almost always be accomplished without interruption to the business of the adjoining tenants, and it is not, therefore, usual to disturb the walls above the basement area. New foundations are built while the superimposed walls are supported by shoring or underpinning.

Shoring.—Methods of shoring will depend largely upon the loads to be carried, the conditions at the building site, and

upon local custom or practice. In some localities, where firm foundation may be had in the new building site, and where the load to be carried is not too great, inclined timber shores are used. These are firmly supported and wedged at their lower ends, and leaned against the wall to be supported at an angle of 10° or 15°, and from their upper ends are suspended hanger-rods, with adjustable turn-buckles, and with large flat hooks or arms at the lower ends which hook through slots or openings cut in the wall at sufficient intervals to provide adequate support.

Another ordinary method is through the use of needle-beams, these consisting of rails, beams, or wooden girders, laid through openings cut in the wall near its base. The needle-beams are supported at either end on cribs or blocking (usually adjusted by means of jack-screws), their distance centre to centre being sufficiently small to carry the wall safely over the intervening spaces. The wall is firmly wedged over each beam, so that it is properly supported while the lower portion is removed to permit the construction of the new foundation. The new wall is built up to the under side of the supported portion, the joints being well wedged, and after the mortar has well set, the needle-beams are removed and the holes are filled up.

In some instances, notably in the building of the foundations for the American Surety Co.'s Building, New York, where room in the new building site was badly needed, the continuous row of cribwork or blocking under the needle-beams was replaced by a truss built directly against the old wall, from which truss the needle-beams were suspended by means of adjustable rods. Each end of the truss was supported on crib-work and jacks, but the intervening space was thus left free and open for work.* In later cases, small groups of piles

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* For complete description, see Trans. Am. Soc. C. E., vol. xxxvii. p. 42.

which occupied very little ground space were substituted for the cribwork, thus leaving the new lot comparatively unobstructed.

During the construction of the Standard Oil Co.'s Building, New York, the side wall of a five-story building, estimated to weigh about 9 tons per lineal foot, was supported by means of wooden needle-beams as shown in Fig. 166.* The inner ends

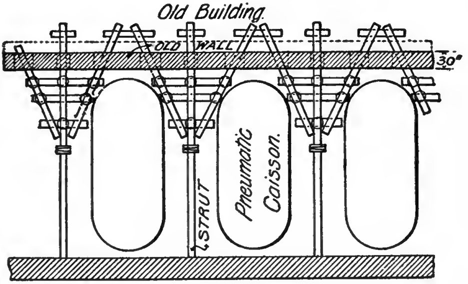

Fig. 166.—Needle-beams used in Shoring at Standard Oil Co.'s Building.

rested on timber blocking, while the outer ends were carried directly on clusters of piles driven within the site of the new building; but as it was necessary to leave vacant the spaces for new pneumatic caissons, and as single needle-beams between these spaces would be too far apart to support the intermediate wall, the piles were arranged as shown, capped with timbers parallel to the wall, and supporting radiating needle-beams.

In some cases the shoring of the old building which was to remain has been started before the demolition of the buildings to be removed. This is done from the basement of the building to be torn down, and is a saving of time, often of considerable importance.

* See the *Engineering Record*. vol. xxxviii. No. 1.

Underpinning.*—When deep excavations, such as the pneumatic type or heavy piling, must be conducted for a new and important structure alongside an adjoining building of great weight but of unsatisfactory foundation construction, unquestioned support from either hard-pan or bed-rock is often desired, and in such cases the ordinary methods of shoring are impracticable. Underpinning from rock or other reliable material is now very frequent in important building operations, even where hard-pan or rock is found only at very considerable depths; and this practice has served greatly to lessen the dangers and difficulties of placing foundations for high buildings under the very severe conditions imposed by adjacent structures.

The method of underpinning now employed insures the rigid support of the adjoining building, "thereby avoiding the usual, though often small movements which follow the removal of the artificial supports used during the period of construction," and also the freedom from obstruction of the site to be built upon.

The operation of underpinning, as employed on the buildings adjacent to the Commercial Cable and Queen's Insurance buildings, New York, may be briefly described as follows: (See Fig. 167.)

Vertical slots are first cut into the wall to be supported, one over each supporting pipe, the length being usually 10 or 12 ft., and the width sufficient to receive a pipe of the diameter calculated as necessary for support. Transverse or cross slots are then cut at the top of each vertical slot, into which one or more steel beams are placed and firmly wedged to support the wall. A length of iron pipe is then placed within a vertical

* For a complete description of this subject, see "The Underpinning of Heavy Buildings," by Jules Breuchaud, in Trans. Am. Soc. C. E., vol. xxxvii.

slot, and a jack and blocking are inserted between the top of the pipe and the short I-beams already inserted. The pipe is then driven into the ground, either by pressure from the jack, or by aid of a water-jet, until a second section can be added on top of the first by means of screw couplings, or interior bolted flanges. By alternate jacking and blocking, this opera-

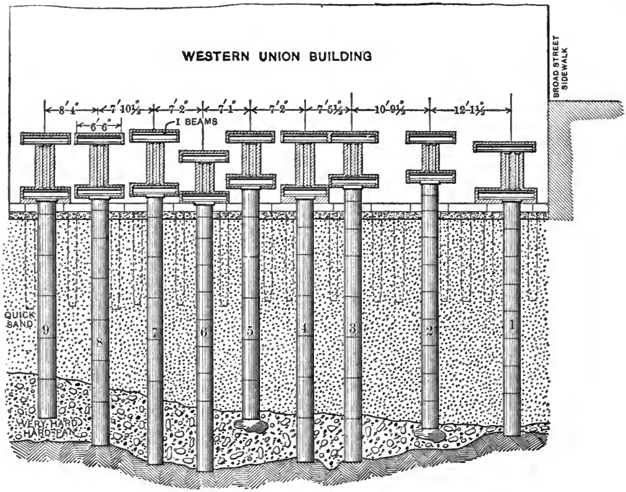

FIG. 167.—Underpinning at Commercial Cable and Queen Insurance Co.'s Buildings.

tion is continued until bed-rock or other satisfactory material is reached.

The top of the last section of pipe driven is left at about the level of the bottom of the wall, and a second set of horizontal I-beams is then placed directly on top of the pipe, as shown in Fig. 167. Vertical beams or columns are then tightly driven between the two sets of I-beams, and the slot in the wall is filled in with brickwork. The compression of

the mortar-joints in the brickwork so built in is thus avoided. One or two pipes only are driven at one and the same time.

For the support of the Western Union Building, see Fig. 167, nine pipes were used to support a side wall 57 ft. long. The pipes were heavy steam-piping, 10 ins. diameter and $\frac{3}{8}$-in. metal, in lengths of 5 ft., and connected by outside couplings over butt-joints. Each alternate pipe enclosed a smaller interior one, placed so as to break joints, the space between the two being grouted with Portland cement. After the pipes were driven to hard-pan or bed-rock, they were filled with Portland-cement concrete.

Two distinct systems of working have been employed. First, the small-pipe system, in which the diameter of the pipe is too small to permit of an inspection of the bottom, and where the tubing is driven to refusal, or until the pressure exerted by the jack is greater than the final load to be carried by the pipe. This method can only be used under favorable conditions, for small pipes are only reliable when driven to hard-pan or rock. The striking of a bowlder might indicate a sufficient resistance, and yet quicksand under the bowlder might be drawn off under the sinking of nearby caissons, and allow a subsequent settlement of the pipe. To be sure of the absence of bowlders, preliminary test borings should always be made.

The pipes are generally made strong enough to support the required load before being filled with concrete. If steel or wrought-iron pipe is used, the outside surface is unprotected, and ultimate destruction through corrosion cannot be prevented; while cast-iron, which is considered less liable to rust, is more unreliable under the jack pressure. It has been suggested first to force down a thin steel pipe, and then place a cast-iron pipe within, filling the intervening space with grout.

The second system of working is through the use of cylinders large enough in interior diameter to permit of reasonably comfortable access, both for working and for examination. If

the final load on the cylinder is larger than can be exerted by means of jacking, or if test borings show that obstacles exist, and the location cannot be changed, large cylinders must be employed. For this purpose 28-in. and 33-in. diameter cast-iron columns have been used, of 1½-in. metal, where the pipe was extended to rock bottom or below a stratum of hard-pan which had to be removed by excavating the hard-pan from around the lower edge of the pipe. Bowlders were also removed, and the rock surface prepared for proper bearing. In such cases, work is usually done under air pressure, an air-lock being attached to the top of the pipe, and sufficient air pressure supplied to keep the pipe free from water.

Cast cylinders of this type, 30 ins. in diameter, with a sectional area of 91 sq. ins. metal, have been loaded with from 200,000 to 380,000 lbs., and some as high as 686,000 lbs., thus giving loads at the foot of the cylinders of 41,200 lbs. per sq. in. and upwards, the metal in the cylinders being in general strained by a compressive force of not over 4,000 to 5,000 lbs. per sq. in.

Settlement.—In constructing foundations upon compressible soils, great care must be given to the question of settlement. If piling is used, driven to bed-rock or hard-pan, or if pneumatic foundations are sunk to bed-rock, no special thought need be given to settlement, as none should occur if the foundations have been properly designed and executed; but for all forms of grillage or raft foundations, resting directly upon earth or clay, the question of settlement is very important.

The danger to be guarded against is *unequal* settlement, for settlement in some degree is sure to follow, and in good design this is anticipated and provided for in fixing the original grades; but if the various foundation piers settle at all unequally, the cracking and separating of the component parts will result in unsightly blemishes, if not in dangerous strains upon the masonry or steel construction.

The evil of unequal settlement can hardly be better exemplified than in the case of the former United States Government Post-office and Custom-house in Chicago, built in 1877, and now being replaced by a new one. The foundations consisted of a continuous sheet of concrete, made in different layers, but altogether 3 ft. 6 ins. thick. Some portions of the building were extraordinarily heavy, others comparatively light, but the concrete base was thought to be sufficient, even though there were bad sloughs under the building. But it proved a most dismal failure, and even a menace to life and limb. The building settled nearly 24 ins. in places, and a dropping of some part of the structure was no unusual occurrence. After but eighteen years of service this example of government architecture and engineering was known as "The Ruin" in Chicago and vicinity.

In Europe, numerous examples exist of a similar monolithic concrete construction under heavy buildings, but in all such cases the concrete base is exceedingly thick, and this thickness is relied upon to resist the uneven reactions from the uneven pier- and wall-loads. The Nicolas Church in Hamburg is said to rest upon a bed of concrete 8 ft. thick, while under the tower, the concrete base is 11 ft. 6 ins. thick.

When a load is applied to the surface of a clayey soil such as exists in Chicago, an initial settlement occurs at a pressure of about 1 ton per sq. ft. Another settlement, which ceases in a few hours, is produced under an increased weight, and further settlement will not directly occur even with a load of 4,500 lbs. to 3 sq. ft. There is, however, a further progressive settlement, owing to the gradual pressing out of the water from the clay. Baker says: "The bearing-power of clayey soils can be very much improved by drainage, or preventing the penetration of the water." That the water is pressed from the clay was shown to be the case by careful observations made at the Auditorium. Wells were sunk some 24 ft. deep, 5 ft.

in diameter, and 4 ft. 6 ins. from the foundations. The borings were made through the stratified clay, and it was shown that the clay became more and more compact from time to time, thus proving that this squeezing process does take place. The settlements were here carefully watched for a number of years, and they were found to be uniform—about $\frac{1}{16}$ in. per month.

If the building is heavy, an immediate settlement of from $2\frac{1}{2}$ ins. to 4 ins. is noticed, followed by a gradual progressive settlement. The Monadnock Building, 200 ft. high, with 3,750 lbs. per sq. ft. on footings, settled 5 ins., while 6 ins. was allowed for. The Home Insurance Building settled $\frac{3}{4}$ in. under two stories which were added to the original building. The Y. M. C. A. Building, with a foundation load of 3,500 lbs. per sq. ft. on the clay, settled 2-$\frac{11}{16}$ ins. in two years; but as this building was erected very slowly, covering about two years from start to finish, the settlement was no doubt considerably lessened.

To ascertain the settlement of the Masonic Temple, Chicago, levels were taken for various columns in this structure, the readings extending over a period of five years, or from 1891 to 1895, inclusive. Some of these settlements have been plotted graphically,* and the results show that the "curves are rapidly approaching a horizontal line; the amount of settlement since the last levels were taken, almost two years before, is nearly the same in each case, the maximum variation being only $\frac{1}{8}$ in., although they had varied considerably before." *

The four exterior corner columns in this building showed total settlements after five years of $7\frac{7}{8}$ ins., $8\frac{9}{16}$ ins., 11 ins., and 8-$\frac{9}{16}$ ins., respectively.

In good design, the anticipated settlement is provided for in the start by raising the level or grade of the footings by the

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* See E. C. Shankland in Proc. of the Institution of C. E., vol. cxxviii. Part II.

amount it is expected the structure will settle. This sometimes causes the sidewalks to slope rather steeply from building-line to curb, but as the building settles, more level conditions obtain. The footings of the Great Northern Theatre (D. H. Burnham & Co., architects) were raised 9 ins. to provide for this amount of settlement.

It must not be forgotten that the footings are designed for the *final* loads that rest upon them, and at all stages of the construction the same relation must be maintained between the weights on the various piers that will exist in the completed state, if uniform settlement is desired. This was well exemplified in the case of the Auditorium tower, which extends many stories above the main building, thus bringing greater weights on the tower footings. Here the tower foundations were loaded with varying weights of pig-iron at the different stages of construction, in order that the proper relative excess on these piers should be preserved as in the final weight. Even with all these precautions, and after most careful tests of the ground beforehand, this tower has settled more than originally allowed for, or more than 20 ins., but this was partly due to adding several stories to the height of the tower, after the foundations had been completed.

Concrete.—The employment of considerable quantities of concrete, in some form or other, is now so general in foundation construction that the proper composition and method of using concrete enter into nearly all large building operations.

In most cases of grillage foundations, concrete is used for the bottom or bed-course, as shown in Figs. 171 and 174, the thickness generally varying from 1 to 2 ft.; but, in special cases, concrete has been applied in a continuous sheet over the entire building site, as a protective layer or covering over a less reliable material below. The site of the St. Paul Building, New York, where the natural surface consisted of a dense wet sand, was thus covered with a uniform layer of concrete

12 ins. thick, upon which were placed the grillage foundations of steel beams. Concrete is also sometimes used in piers, and for the encasement or protection of grillage members, cantilever girders, etc., as well as for filling the interiors of pneumatic caissons and air-shafts.

The composition of concrete is sometimes specified by building ordinance. Thus the New York law requires foundation concrete to "be made of at least one part of cement, two parts of sand, and five parts of clean broken stone," or "good clean gravel may be used in the same proportion as broken stone." The Chicago Ordinance does not specify the proportion of the ingredients.

In general, it may be stated that the best results are obtained from compositions in which the volume of the mortar is slightly in excess of the voids or spaces between the loose broken stone or gravel employed. For material of average uniform size, the voids will run about 40 to 50 per cent. of the mass. The stone should be clean and screened, and of such size that it will pass through a 2-in. diameter ring in any direction. The sand must be sharp and clean, and the cement fresh and dry, these materials being mixed dry, with sufficient water added to reduce the mass to the consistency of mortar. The concrete should be laid in layers not over 6 or 8 ins. thick, and rammed until water shows at the surface.

The concrete usually specified for U. S. Government work is 1 part cement, 3 parts sand, and 5 parts broken stone. This or similar mixtures may be considerably cheapened without materially affecting the strength by using about equal parts of broken stone and clean gravel instead of the 5 parts stone. Prof. Baker states that the concrete foundations under the Washington Monument were made of 1 part Portland cement, 2 parts sand, 3 parts gravel, and 4 parts broken stone, and that this mixture stood, at 6 months old, a load of 2,000 lbs. per sq. in., or 144 tons per sq. ft. The concrete used in

the Masonic Temple foundations was made of 1 part Portland cement, 2 parts clean sharp torpedo sand, and 3 parts clean stone broken to pass a $2\frac{1}{2}$-in. diameter ring.

The safe bearing loads on concrete, not reinforced by metal members, is limited to 4 tons per sq. ft. by the Chicago Building Law (the offsets to be not more than one-half the heights of the respective courses), and by the New York laws to 15 tons per sq. ft. when made of Portland cement, and 8 tons per sq. ft. if cement other than Portland is used.

The crushing strength of concrete varies greatly with the time it has been set, as the strength rapidly increases with age. Average ultimate crushing strengths for good concrete may be placed at about 15 tons per sq. ft. for 1 month old, 60 tons for concrete 6 months old, and 100 tons for an age of 12 months. Assuming a safety factor of six, working loads would become $2\frac{1}{2}$ tons for concrete 1 month old, 10 tons for concrete 6 months old, and 16 tons for concrete 1 year old. These strengths would suggest the desirability of placing concrete foundations as early in the building operations as practicable; but the superstructure weights are increased gradually, and the foundations are almost invariably in place several weeks before any great load is applied. It is usually 4 months at least before the full load is reached, so that the concrete has ample time to set.

When used between beams of grillage foundations, the stone employed must be broken fine enough to allow of ramming in between the beam webs and flanges. For such cases, the stone is usually specified to be broken to pass through a $\frac{3}{4}$-in. ring, or broken to "chestnut" size. Crushed granite is also used, not exceeding $\frac{1}{2}$-in. cube.

Foundation Loads.—In all cases where live-loads have been figured on the columns, consistency requires that whatever loads have been figured on basement columns, must be figured in the calculations of the foundations; or the bearing

areas are proportioned for dead-loads only, while the strengths of the foundations themselves are figured for dead- plus some live-load. But, as before said, live-loads have been entirely disregarded on the footings of many of the best buildings. W. L. B. Jenney advocates as follows: In hotels, office buildings, and retail stores, neglect the live-loads on the footings, but figure them in heavy warehouses, machinery plants, etc. Where much pounding occurs, as in machinery in motion, use double the weight as dead-load that is figured for live-load.

In "The Fair" Building, where a large quantity of merchandise is stored, and the aisles are constantly filled by throngs of people, the following system was used: The floor-beams carry all the dead- plus live-loads, the girders carry the dead-load plus 90 per cent. of the live-load, while any one column carries a percentage of the sum of the live-loads of all the stories above that column plus the total dead-load. The percentage of live-load is given in the last column of the accompanying table:

Column.	Live-load on beams.	Per cent. for column.
Attic	——	100 per cent.
16th story	75 lbs.	90 " "
15th "	75 "	87½ " "
14th "	75 "	77½ " "
13th "	75 "	72½ " "
. . . . . .	. . . . . .	Decrease of 2½ per cent. in each story.
6th story	75 lbs.	55 per cent.
5th "	130 "	52½ " "
Basement	130 "	40 " "

No live-load was figured on the clay area, but the allowable pressure per square foot was taken at a very conservative figure—2,850 lbs.

As before stated, the loads to be used in proportioning foundations are often specified by building ordinance.

The New York code provides as follows for loads to be used in designing footings in buildings more than three stories in height:

"For warehouses and factories they are to be the full dead-load and the full live-load established by this code.

"In stores and buildings for light manufacturing purposes they are to be the full dead-load and seventy-five per cent. of the live-load established by this code.

"In churches, schoolhouses, and places of public amusement or assembly, they are to be the full dead-load and seventy-five per cent. of the live-load established by this code.

"In office buildings, hotels, dwellings, apartment houses, lodging houses, and stables, they are to be the full dead-load and sixty per cent. of the live-load established by this code.

"Footings shall be so designed that the loads will be as nearly uniform as possible and not in excess of the safe bearing capacity of the soil, as established by this code." (See *Bearing Pressures*, *Building Laws*.)

The Chicago ordinance makes no specific requirements as to foundation loads, but states that "foundations shall be proportioned to the actual average loads they will have to carry in the completed and occupied building, and not to theoretical or occasional loads."

For working under such requirements as the New York law, methods are given in Chapter VII. under a discussion of column sheets, etc., whereby the dead- and live-loads may be kept separate, and hence conveniently selected for the computations of the foundations.

Present Types of Foundations.—The various methods of securing adequate foundation areas for the loads to be supported may be classified as follows:

1. By simply building the walls or piers upon the natural

soil, the necessary base being secured by means of projecting courses of masonry. This method is only applicable to buildings of very moderate height and load, and need not be here considered in detail. The only requirements demanding special attention are that the soil must be of the required bearing capacity, that the bed of the foundation shall be below the frost-line, and that the centre of pressure must always coincide with the centre of base.

2. By obtaining the necessary bearing area by means of timber platforms or grillage, as was utilized in the construction of the World's Fair buildings, and in the Chicago Auditorium. This method will be more fully explained under a following heading.

3. By utilizing a grillage composed of steel rails, beams, or riveted girders in combination with concrete. This type is often used to support two or more columns upon one grillage, in which case the footing is termed a "combined footing."

4. By driving piles to some hard or firm material, usually designated hard-pan, or to rock.

5. By sinking steel cylinders, or caissons of timber or steel (by the pneumatic process or otherwise), to bed-rock, or to such material as will answer the purpose of bed-rock. This system is also used to support either a single column, or a number of columns.

Types 3, 4, and 5 will each be considered in detail in following paragraphs. Forms 3 and 5 are also often used in connection with cantilever girders, but the introduction of such girders makes but a variation in the detail of the calculations.

Proportioning Grillage Areas.—An investigation of the compressibility of the soil leads to the conclusion that, if we wish to procure uniform settlement, all parts of the foundation areas must be exactly proportioned to the loads they have to carry. Examples are not lacking, in Chicago and elsewhere, of the actual crushing of light piers, when alternating with

heavy ones, because, *proportionately*, the lighter piers had too great a footing area. In the Mills Building in New York City the mullions in the lower floors of the building and over the light foundations were seriously damaged and even crushed, because they were not strong enough to force down the lighter piers of too large an area, as fast as the heavy piers were settling.

It is the judgment of the best engineers that the areas of foundations on compressible soil should be proportioned to the dead-loads only, and not to theoretical or occasional loads. Whenever live-loads have been figured on both the interior columns and on the columns in the exterior walls, the exterior columns have always been found to settle more, from the fact that the live-load forms a larger percentage of the interior-column loads than of the wall-column loads.

Thus in the Marshall Field warehouse in Chicago, designed by an eastern architect, the live-load of 75 lbs. per square foot on every floor was carried down to the footings, according to the then-prevalent custom in New York and Boston, the result being that all of the floors have risen considerably at the centre.

Experience has also shown that after the clay has been compressed by a load of 3,000 lbs. per sq. ft., and allowed several months' repose, no very perceptible addition to that compression will result without a material addition to the load. It is therefore good practice to neglect live-loads on the clay for hotels, office buildings, or lightly loaded retail stores, if permitted by the local building laws. In warehouses, however, or in buildings carrying very heavy permanent or shifting floor-loads, or machinery in motion, the change of loads and the jarring increase the compression of the clay very largely. Hence extra allowance must be made in such instances. This is sometimes done, for grillage foundations, by proportioning the foundation area for the pier receiving the maximum com-

bined dead- plus live-loads, for the allowable unit bearing pressure. Then, assuming that only the dead-load acts over the area so found, compute the resulting unit bearing pressure, and use this unit in proportioning the remaining piers for dead-loads only. Thus, assume that the maximum column load is 400 tons dead-load, plus 240 tons live-load, or 640 tons total. Assuming, also, a unit bearing pressure of 4 tons per sq. ft., the foundation area required is $\frac{640}{4}$ or 160 sq. ft. Considering now that only the dead-load acts, the foundation area will receive but $\frac{400}{160}$ tons per sq. ft., or $2\frac{1}{2}$ tons per sq. ft. The remaining footings may then be proportioned by dividing the dead-loads only by the unit of pressure, $2\frac{1}{2}$ tons per q. ft., as determined above.

The method of proportioning the grillage areas, however, will be largely governed by municipal regulations, as explained under the heading "Foundation Loads."

Timber Grillage.—For temporary work, or for spread foundations in very wet soil, timber grillage may be employed to increase the bearing areas of the footings, provided the timber employed is always below the water-line. The use of this method, however, is not to be recommended for loads of any considerable magnitude in permanent structures, as steel and concrete grillage may be substituted to advantage. Steel grillage will permit of greater offsets, and also require less thickness for the footings, than may be obtained through the use of timber construction. Also, steel and concrete are not dependent upon conditions of moisture for use, while timber, to insure preservation against rapid decay, must be kept wet at all times.

A notable example of temporary timber grillage was the use of this system in the buildings of the World's Columbian Exposition at Chicago. It was first intended to use pile foundations throughout, but at the same time as driving test piles, test platforms of 3-in. plank were constructed and placed in

different locations upon the sandy soil, for the purpose of testing the surface bearing capacity. These were then loaded with pig-iron to about 2¼ tons per sq. ft. This load was applied gradually, and the settlements were carefully noted for several days. It was found that under the maximum loading the settlement was very slight, about ⅜ of an inch, and very uniform. It was therefore decided to use pile foundations driven to hard-pan in certain locations where quicksand existed, but to use platform foundations proportioned for a bearing pressure of 1¼ tons per sq. ft., where the ground was favorable.

The general design of these platform foundations was as shown in Fig. 168. They consisted of 3-in. pine or hemlock

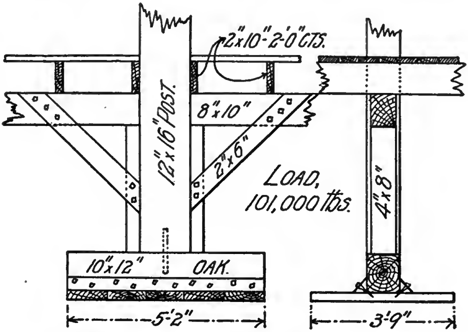

Fig. 168.—Timber Grillage Foundation, Fisheries Building, World's Columbian Exposition.

planks, with blocking on top to distribute the load uniformly over all the planks. This blocking also served as a support for the posts, which carried the caps and thus the floor-joists and upright posts of the building. The blocking was well spiked to platform planks and posts, and caps and sills were drift-bolted. For bearing of vertical posts upon underlying blocking (end fibre upon transverse fibre), a unit pressure of

400 lbs. per sq. in. was allowed. For tension in extreme fibres of caps and joists, a unit of 1,200 lbs. per sq. in. was used.

The footings under the New Orleans Custom-house are mentioned by Prof. Baker (see "A Treatise on Masonry Construction," p. 211), as an example of timber grillage. Upon a plank floor laid 7 ft. below the street-level, were placed 12-in. diameter logs, side by side, over which similar logs were placed transversely, 2 or 3 ft. apart. The open spaces were filled with concrete, and a continuous layer 1 ft. thick was then spread over the entire area. The settlement has been very great, and far from uniform, but had the same foundation materials been employed in independent footings, proportioned for the separate pier-loads, unequal settlement would have been avoided.

For allowable offsets in timber courses, Prof. Baker gives the following: "If the pressure on the foundation is 0.5 ton per sq. ft., the safe projection is 7.5 times the thickness of the course; if the pressure is 1 ton per sq. ft., the safe projection is 5.3 times the thickness of the course; if the pressure is 2 tons per sq. ft., the safe projection is 3.7 times the thickness of the course. The above values give a factor of safety of about 10."

Masonry in Foundations.—The application of masonry to foundation design will usually be in the form of cap-stones over piles, as in Fig. 184, or as masonry piers of brick or stone over pile or caisson under-bearings, as in Figs. 183 and 194.

Data as to the compressive strength of various classes of masonry, also unit-stresses and the requirements of building laws have been previously given in Chapter V., but for the use of masonry construction in foundations, it will be necessary to determine the allowable batters for brickwork or masonry of dimension-stones, also the allowable offsets or thicknesses for dimension-stones where used as cap-stones or in stepped-out foundations.

The following table* gives the safe offsets for masonry footing courses in terms of the thickness of the course for a factor of safety of 10:

Kind of Stone.	Offset for a Pressure, in Tons per sq. ft., on the Bottom of the Course of		
	0.5	1.0	2.0
Blue-stone flagging	3.6	2.6	1.8
Granite	2.9	2.1	1.5
Limestone	2.7	1.9	1.3
Sandstone	2.6	1.8	1.3
Slate	5.0	3.6	2.5
Best hard brick	2.7	1.9	1.3
Hard brick	1.9	1.4	0.8
Concrete { 1 part Portland cement, 2 " sand, 3 " pebbles } 10 days old	0.8	0.6	0.4
Concrete { 1 part Rosendale cement, 2 " sand, 3 " pebbles } 10 days old	0.6	0.4	0.3

Results given by the above table are correct only when the footing is composed of entire stones for each course, and when the projections are not more than half the lengths of the stones.

"The preceding results will be applicable to built footing courses only when the pressure above the course is less than the safe strength of the mortar. The proper projection for rubble masonry lies somewhere between the values given for stone and those given for concrete. If the rubble consists of large stones well bedded in good strong mortar, then the values for this class of masonry will be but little less than those given in the table. If the rubble consists of small irregular stones laid with Portland or Rosendale cement mortar, the projection should not much exceed that given for concrete. If the rubble is laid in lime mortar, the projection of the footing course should not be more than half that allowed when cement mortar is used."

* See Prof. Baker's "Treatise on Masonry Construction," p. 209.

For offsets in brick piers, both the New York and Boston building laws require that if the bricks are laid in single courses the offset for each course must not exceed 1½ ins., or, if laid in double courses, then each offset shall not exceed 3 ins.

The allowable loads and offsets for concrete piers under the Chicago building laws are given under the preceding heading "Concrete."

The Chicago ordinance specifies that the offsets in dimension-stones, where two or more layers are used, "must not be more than three-quarters of the height of the individual stones," and that "dimension-stones in foundations shall not be subjected to a load of more than 10,000 lbs. per sq. ft. in piers. If the beds of the stones are dressed and levelled off to uniform surface and the stones are set in cement mortar, this strain may be increased to 14,000 lbs. per sq. ft."

Comparison of Masonry Offsets and Steel Grillage.—The rapid development of foundation design is well exemplified by the great change in methods employed at the site of the Woman's Temple, Chicago. In 1890 the lot where this building now stands was bought by the present owners. Extensive masonry foundations had been built here a few years before for a structure that was never erected, and upon the preparation of plans for the Temple, the first thing done was to remove these massive masonry piers at great cost. The old system consisted of stone piers made of successive layers of large stones, stepping out until a sufficient base was obtained. One of the newer "raft" footings is shown in Fig. 169, and also one of the old masonry type, in Fig. 170.

The objections to these old piers were many: they were bulky, occupying too much space; they were heavy and costly as regarded the time necessary for building; and the allowable offsets of the masonry work seriously limited the load-bearing surface of the clay.

These piers in the Woman's Temple were all underlaid

with a bed of concrete, resting on the clay stratum about 10 ft. below street-grade. A comparison of some of the above points may be made as follows:

I. *Space.*—

1st. Top of concrete to bottom of casting = 1′ 8″.

2d. " " " " " " " = 7′ 0″.

Or, comparing the parts above the common bed of concrete,

1st = 217 cu. ft., 2d = 691 cu. ft.

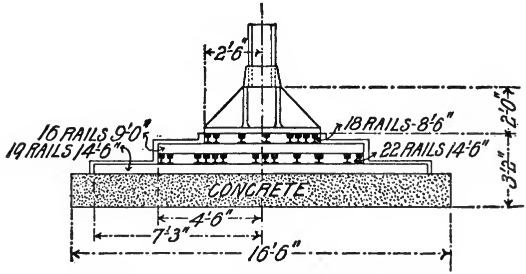

FIG. 169.—Detail of Rail-grillage Footing.

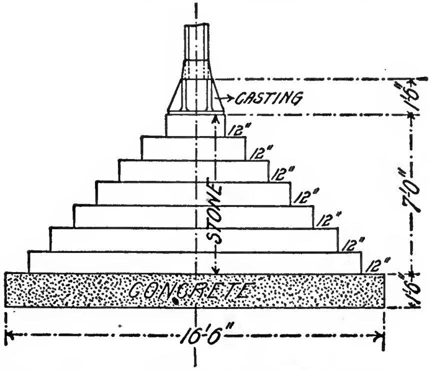

FIG. 170.—Detail of Masonry-pier Footing.

This point of space is a very important one, as has been before mentioned, since basement-space is now quite as valu-

able as any office-space, for use as restaurants, cafés, or for the large boiler and electric-light plants necessary. It is even of frequent occurrence to extend the basement-space out under the sidewalks and alleys. Thus, to gain cellar-room, the foundation must either be lowered or made thinner. The first has been ruled out of Chicago practice, because it has been clearly demonstrated that the less the clay stratum is broken, the more uniform and satisfactory the settlement will be.

II. *Weight.*—Rating the masonry at 150 lbs. per cu. ft., concrete at 140 lbs., and allowing 44 cu. ft. for the steel in No. 1, the weights are:

No. 1 = 103,000 lbs.
No. 2 = 261,000 lbs.

The load for this foundation is 800,000 lbs., and the saving in weight, through the use of the raft foundation, is thus sufficient to allow an additional story, without adding to the load on the clay. On large foundations this difference is still greater. In the case here assumed the 103,000 lbs. is about 13 per cent. of the load carried, but in some cases, under very heavy loads, it has been found to run as high as 20 per cent. (see "Steel Rail Foundations," *Engineering News*, vol. xxvi. No. 32.

This saving in weight is one of the factors that makes our highest buildings possible, and fourteen or even sixteen stories are not loading the clay as severely as some of the older structures. Under the foundations of old five- or six-storied masonry buildings, which were torn out to make room for new office buildings, the clay has been found loaded to 11,200 lbs. per sq. ft., while "The Fair" Building is loaded to 2,850 lbs. only, for a sixteen-story modern structure.

III. *Cost.*—In general, the cost of stone foundations will be less than iron ones, but considering the renting-space in basements, this difference will be quickly made up where the latter are used.

IV. *Time.*—In the time required for building operations

the new foundations are greatly superior, as rails and beams are easily obtained and cheaply handled.

V. *Load-bearing Area.*—As to the fifth point, stone foundations under side walls frequently cannot step out sufficiently to get the proper bearing area without projecting into the next lot. But with steel we can combine several footings, or use cantilever foundations, thus securing the desired results. For interior footings, also, it would be difficult, in fact practically impossible, to obtain sufficient bearing areas for great loads through the use of masonry offsets, except with great height and attendant bulk of material.

Rail Footings.—The raft footings as first employed were made of rails only, the usual method of figuring being as follows: The number of square feet of footing required equals $\frac{\text{load on column}}{\text{pounds per sq. ft. on earth}}$. Multiply the result by 250 (equals approximate weight of footing per square foot), add to the original load, and refigure. The layers were then laid off, the projection of any layer beyond the one immediately above being always 3 ft. or less. The moments on the projecting portions of the layers were then found, and these moments, divided by the allowable bending moment per rail, usually taken at 12,500 ft.-lbs., gave the number of rails required in the different courses. One extra rail was usually added to each layer as a matter of safety.

The following table gives the properties of the rails from the North Chicago Rolling Mills. The 75-lb. rails were most commonly used.

No.	Weight.	Height.	Base.	*u*.	*I*.	*R*.	*M*. $f = 16{,}000$.
6504	65 lbs.				15.86	6.86	9,150
7501	75 "	$4\frac{3}{4}''$	$4\frac{1}{2}''$		21.00	8.30	11,070
7503	75 "	$4\frac{3}{4}$	$4\frac{3}{4}$	$2\frac{47}{64}''$	21.66	9.37	12,500
8001	80 "	5	5	$2\frac{21}{64}$	26.36	9.99	13,320
8501	85 "	5	$4\frac{3}{4}$	$2\frac{5}{8}$	27.32	10.41	13,880
8502	85 "	$5\frac{8}{16}$	5	$2\frac{5}{8}$	29.22	11.13	14,840
8503	85 "	5	5	$2\frac{17}{32}$	25.38	10.03	13,370

The table following is taken from the footings of the Great Northern Hotel, giving the loads on columns, areas of footings, and the calculated weights per square foot of the rails and the concrete in the footings. All rails were 75-lb. rails, No. 7503 in the previous table. The bottom courses of all footings were of concrete, 12 ins. thick, extending 6 ins. beyond the lower course of rails, but the weights of these concrete courses are not included in the following. Cast shoes 4 ft. × 4 ft. were used under all the columns. The concrete was figured as weighing 125 lbs. per cu. ft.

Load on Col.	Area of Footing, sq. ft.	Weight per sq. ft.	
		Rails.	Concrete.
415,470	$12 \times 11\frac{3}{4} = 141$	49	83
433,440	$10 \times 14\frac{1}{2} = 146$	58	60
435,820	$9 \times 16\frac{1}{4} = 146$	77	83
461,100	$12 \times 13 = 156$	42	80
496,240	$10 \times 16\frac{1}{3} = 163$	79	91
526,850	$12\frac{3}{4} \times 14 = 178$	66	82
531,740	$13 \times 14\frac{1}{4} = 185$	60	78
571,360	$13\frac{1}{2} \times 14\frac{1}{4} = 192$	67	74
595,920	$12\frac{1}{2} \times 16 = 200$	67	88
621,560	$13 \times 16 = 208$	60	94
637,240	$13\frac{1}{2} \times 16 = 214$	68	68
666,000	$15 \times 15 = 225$	66	105
672,000	$13 \times 17\frac{1}{2} = 228$	67	93

Beam and Rail Footings.—The next step made in the development of raft footings was in the use of I-beams for the upper course or courses. Fig. 171 shows a foundation which was figured as follows (see *Engineering News*, vol. xxvi. No. 32):

The column load was 1,166,000 lbs. The allowable pressure per square foot on the clay was taken at 3,000 lbs., giving a footing 22 ft. 8 ins. × 17 ft. 3 ins. The lower layer of concrete was 18 ins. thick, projecting 8 ins. beyond the lower course of rails. Fifteen-inch steel beams were used in the top course, weighing 50 lbs. per ft. The allowable

moment on each beam equalled 117,700 ft.-lbs. The remaining courses were of steel rails, $4\frac{3}{4}$ ins. high and $4\frac{3}{4}$ ins. base,

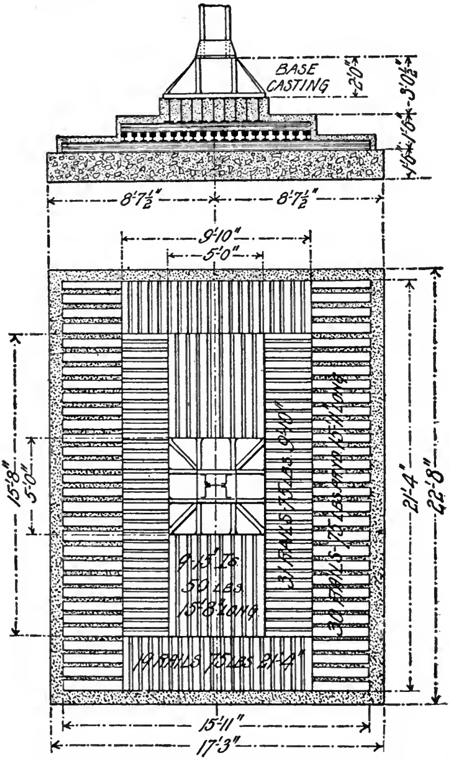

FIG. 171.—Detail of Beam and Rail Footing in "The Fair" Building.

75 lbs. per yard weight, with an allowable moment of 12,100 ft.-lbs. per rail.

In the upper course, as many beams are used as the space under the column casting will allow. The projecting arms must therefore be determined. The total length of the I-beams

so found will fix the width of the second course from the top, and the projecting arms must be found for this course as in the first case.

The arms of the lower two courses are fixed by the lengths of the upper ones, and by the dimensions of the clay area; hence the question is, how *many* pieces are required? The formulæ used may be derived as follows:

Let $y =$ projecting arm in any course;
$a =$ width of supporting area;
$P =$ total load on footing;
$M =$ bending moment on one side of the layer.

Then the length of beam or rail $= a + y + y = a + 2y$. The total load on $y = \frac{Py}{a + 2y}$, and since the distribution of the load on every layer is uniform, we have

$$M = \frac{Py}{a + 2y} \times \text{lever arm } \frac{y}{2} = \frac{Py^2}{2(a + 2y)}.$$

In calculating the lower two courses, y becomes a known quantity and M an unknown. In the upper two layers M is given by the number of the beams used and y is unknown.

Considering now the top course, under the base casting, 5 ft. $\times$ 5 ft. in area, we find that nine beams only can be placed under the casting, allowing sufficient space between them for the ramming of the concrete.

M for each beam $=$ 117,700 ft.-lbs. Hence M for the whole layer $= 9 \times 117{,}700$ ft.-lbs. $=$ 1,059,300 ft.-lbs. Then $\frac{1{,}166{,}000y^2}{2(5 + 2y)} = 1{,}059{,}300$, whence $y =$ 5 ft. 4 ins. The length of this layer then becomes $5 + 2y =$ 15 ft. 8 ins.

For the second course we find that 31 rails spaced about 6 ins. centres may be placed under the 15 ft. 8 in.-beams. Closer spacing than this may be used if necessary. The load

now equals 1,166,000 lbs. + the weight of the top course (about 19,000 lbs.). Then $\frac{1,185,000y^2}{2(5+2y)} = 375,100$; whence $y = 2$ ft. 5 ins. The length of the rails therefore = 5 ft. + 4 ft. 10 ins. = 9 ft. 10 ins.

For the calculation of the lower courses, we know that the area covered by the bottom course is 15 ft. 11 ins. × 21 ft. 4 ins. This leaves a projection of 3 ft. ½ in. for the bottom course, and a projection of 2 ft. 10 ins. for the next to the bottom layer.

Then for the third or next to the bottom course, we have

$$\frac{Py \times \frac{y}{2}}{a + 2y} = \frac{1,200,000 \text{ lbs.} \times 2\frac{5}{6} \text{ ft.} \times 1\frac{5}{12}}{21\frac{1}{3}} = 225,780 \text{ ft.-lbs.} = M.$$

This moment requires 19 rails to be used in the layer.

For the bottom course,

$$M = \frac{1,220,000 \text{ lbs.} \times 3\frac{1}{24} \text{ ft.} \times 1\frac{25}{48}}{15\frac{11}{12}} = 343,000 \text{ ft.-lbs.}$$

This requires 29 rails, 30 being used for safety.

It will be noticed in the above calculation that the moments have been taken for the projections of the several courses beyond the adjacent supporting layers only. Thus in the figures for the next to the bottom course, as given above, $y = 2$ ft. 10 ins. If, however, the foundation be taken as a whole, and the bending moment on the third course is taken around the edge of the cast base, the same as the top course was figured, we have $y = 8\frac{1}{6}$ ft., or,

$$M = \frac{1,200,000 \times 8\frac{1}{6} \times 4\frac{1}{12}}{21\frac{1}{3}} = 1,920,000 \text{ ft.-lbs.}$$

This must be resisted by the combined moments of the 9-in. I-beams in the top layer, and the 19 rails in the third layer,

or 1,059,300 + 229,900 = 1,289,200 ft.-lbs. This assumption leaves a difference of 630,800 ft.-lbs. which has not been cared for.

Both of the above methods of calculating grillage foundations have been extensively employed. Many engineers and architects advocate considering each layer of beams separately, and thus taking moments for each layer about the centre of the system. This method requires more material than when moments are taken about the edges of the supporting layers, but the excess is considered to offset any possible corrosive tendencies. If the individual layers were piled loose, without being embedded in concrete, this method of moments about the centre for each layer would undoubtedly be theoretically correct, but the action of the concrete filling, with its tendency to bind the beams and concrete together, causes the grillage to act largely as a whole and thus to possess a moment of resistance much greater than the sum of the resistances of the individual layers. This latter method is now generally employed, and will be used in the following calculations for simple and combined grillage systems.

The use of rails in footings has now been succeeded by the employment of beams throughout.

Beam Footings.—In the two preceding paragraphs, the methods of calculation employed for Rail Footings and Beam and Rail Footings have been described, and precisely the same procedure may be followed in the design of Beam Footings as now employed in almost all cases of simple grillage.

In determining the sizes of beams in any layer, care must be taken to leave sufficient clearance between the flanges to admit the concrete which must be rammed in place. If stone, crushed or broken to pass a $\frac{3}{4}$-in ring, be specified, 1 in. as a minimum between the flanges will answer.

To surround and protect the various layers, plank frames or open boxes are made for each course, of sufficient size to

permit a 4-in. concrete covering for the ends and sides of the beams, and a 1-in. protective layer over the tops of the courses. The concrete should be well tamped between the beams, and the whole exterior is then plastered with pure Portland cement mortar, so that no part of the metal-work is exposed. A bed of concrete 18 ins. or 2 ft. thick is placed under all, projecting 6 ins. to 12 ins. beyond the beams.

Fig. 172 shows a simple beam footing used in the Marquette Building for a column load of 920,250 lbs.

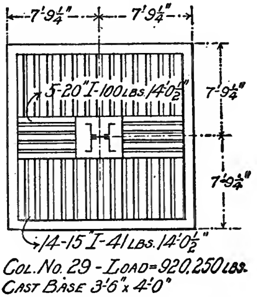

Fig. 172.—Simple Beam Footing, Marquette Building.

Formulæ applicable to this type of footing may be derived as in the following paragraph.

Simple Beam Footings, are those which receive one column only. In this case the concrete bed is made symmetrical about the column, in order that the centre of pressure and centre of base may coincide.

In the following analyses of simple and combined footings, let

P = column load;
a = width of column base;
y = projection of beams beyond base or adjacent layer;
l = length of beams in feet;
f = allowable extreme fibre stress;
S = section modulus.

Referring now to Fig. 173, and considering the upper course of beams of length l, the load per lineal foot will equal $\frac{P}{l}$, and the load at the centre of moments c, will equal $\frac{Py}{l}$.

The bending moment at c will equal

$$\frac{Py}{l} \times \frac{y}{2} = \frac{Py^2}{2l} \text{ foot-pounds.}$$

But, as $S = \frac{M}{f}$, where M equals the bending moment, we have

$$S = \frac{12Py^2}{2lf} = \frac{6Py^2}{lf}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

This value of S is for the total number of beams in the course. The required value of S for one beam will therefore

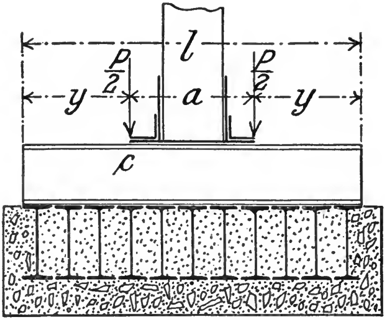

FIG. 173.—Diagram of Calculation of Simple Beam Footing.

be obtained by dividing S as found by equation (1) by the number of I-beams used in the layer, and from the values of S given in the mill handbooks, the required size and weight of beam may be readily selected. For the lower course, the calculation may be made in exactly the same manner, remembering that the point of moments, c, is taken on the extreme edge of the upper course.

Tabulated sizes and weights of beams for simple grillage foundations may be found in the handbook of The Carnegie Steel Co. These are given for allowable bearing capacities of from 1 to 50 tons per sq. ft., and for the spacing of beams 9, 12, 15, 18, and 24 ins. centres.

In order that perfectly accurate bearing may be obtained between the various layers of beams composing a grillage

foundation, some engineers and architects are now specifying that all bearing-surfaces shall be faced or planed. Thus the top and bottom flanges of the I-beams are planed where receiving a layer from above, or where bearing upon a layer beneath. In such cases each separate layer is usually made complete in the shop, the beams being connected by special riveted diaphragms, instead of by the usual cast separators and bolts.

Combined Footings. — In proportioning the areas for adjacent grillage footings, they are often found to overlap, and in such cases two, three, or even four areas may be combined as one footing. When this is done, the centre of gravity of the footing area must coincide with the centre of pressure of the loads carried.

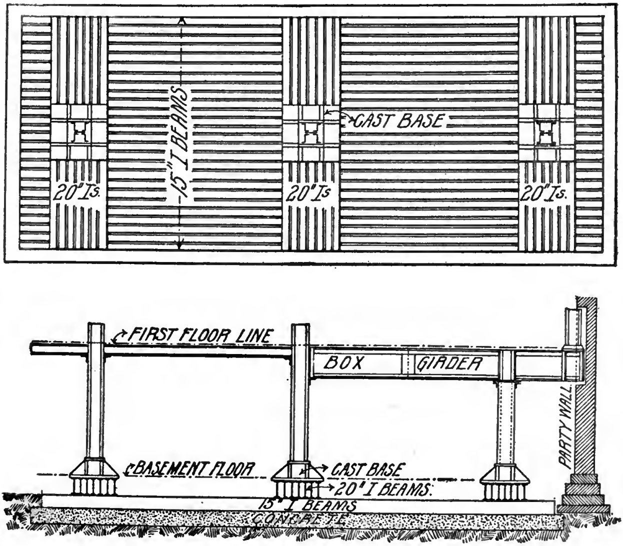

FIG. 174.—Combined Footing, Old Colony Building.

Combined footings are also exceedingly useful where access may not be had to the basement of an adjoining building or buildings, thus precluding the construction of new party-wall foundations by shoring or underpinning. Recourse may then be had to cantilever construction, in which a combined footing is used with cantilever girders to transmit the wall-loads away from the lot-line, and, combining with the other column loads, bringing the resultant centre of pressure over the centre of base.

The first cantilever footings introduced were those in the Manhattan and Rand McNally buildings in Chicago, built at about the same time. The boilers, etc., in the basements of the adjoining buildings, could not be disturbed to allow the introduction of new party-footings, so the cantilever types were adopted for the new structures, and the foundations of the old ones were not interfered with.

Fig. 174 illustrates a combined footing and cantilever girder as employed in the Old Colony Building, Chicago.

Two Equally Loaded Columns, Area Rectangular.—This case may sometimes be met with in very narrow buildings, where all of the columns become outside supports, and with practically the same loads at either side. The top layer beams then become uniformly loaded beams, supported at each end, and the moment for the layer becomes

$$M = 2P \times \frac{l}{8} = \frac{Pl}{4}, \quad \text{and, as} \quad S = \frac{12M}{f},$$

we have

$$S = \frac{3Pl}{f}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The lower beams are calculated by equation (1) as before.

A variation of this case is shown in Fig. 175, where each footing receives four columns, supported on a double set of cantilever plate- or box-girders.

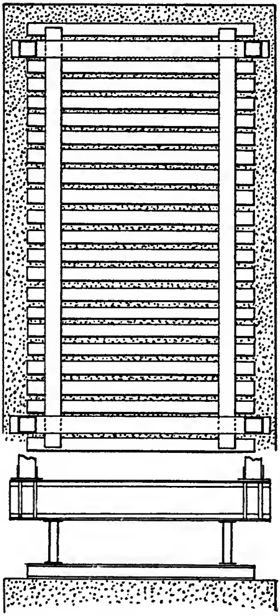

FIG 175.—Combined Footing.

Two Unequally Loaded Columns, Area Rectangular.—This is one of the most ordinary cases in practice. Referring to Fig. 176, the loads P_1 and P_2 are given, also the distance x, centre to centre of columns. A slight inaccuracy is introduced by considering the column centre P_1 as the end of the footing, but as the column base is not usually over 24 ins. wide, the results will be sufficiently exact.

To find the centre of gravity of the two column loads, take moments at P_1. The distance g from P_1 to the centre of gravity will then equal $\frac{P_2 x}{P_1 + P_2}$, and as the total length of the

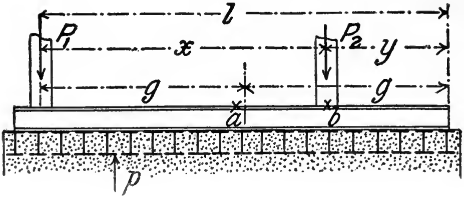

FIG. 176.—Diagram of Combined Footing, Two Unequally Loaded Columns, Area Rectangular.

footing must equal $2g$, in order that the centre of gravity and the centre of pressure may coincide, we have

$$2g = l = \frac{2P_2x}{P_1 + P_2}.$$

The uniform pressure per lineal foot at the base of the footing will equal

$$\frac{P_1 + P_2}{l} = p.$$

To calculate the bending moments, it is first necessary to determine the points of no shear, as the bending moment will be maximum when the shear = 0. Moving to the right from P_1, the first point of no shear, a, is at a distance of $\frac{P_1}{p}$ ft. The second point of no shear will be closely to the left of P_2, or such a distance that enough of P_2 will be added to P_1 to equal px. Sufficiently accurate results will be obtained if this point is considered to coincide with the column centre, b.

Considering now all of the forces to the left of a, we have the column load P_1 and the uniform load on the footing base. The bending moment will equal the moment of the column load, minus the moment of the uniform load, or

$$M = \left(P_1 \times \frac{P_1}{p}\right) - \left(p\frac{P_1}{p} \times \frac{P_1}{2p}\right) = \frac{P_1^2}{2p}.$$

Substituting the value of p previously found, we have

$$M = \frac{P_1^2}{2} \div \frac{P_1 + P_2}{l} = \frac{P_1^2 l}{2(P_1 + P_2)}. \quad . \quad . \quad . \quad (3)$$

For the bending moment at P_2, or the point b, take the moments of the forces to the right of the section. The resultant moment will equal the moment of the uniform load on the footing base, minus the moment of one-half the column load into its arm, or one-fourth the width of the base casting.

Or, $$M = \left(py \times \frac{y}{2}\right) - \left(\frac{P_2}{2} \times \frac{c}{4}\right),$$

where c = width of base casting.

$$\therefore M = \frac{py^2}{2} - \frac{P_2 c}{8},$$

and substituting the value of p, as before, we have

$$M = \frac{(P_1 + P_2)y^2}{2l} - \frac{P_2 c}{8}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The required section modulus, or S, may then be determined by substituting the greater of the two moments thus found by equations (3) and (4) in the equation

$$S = \frac{12M}{f}, \text{ as before.}$$

The beams in the lower course may be found by equation (1).

Two Unequally Loaded Columns, Trapezoidal Area.— When two unequally loaded columns are to be supported upon one footing, and one of the columns, probably the heavier, is a wall column whose foundation may not extend beyond the lot-line, a trapezoidal footing area can be used instead of the rectangular bed previously calculated.

As before, the centre of gravity of the loads must coincide with the centre of pressure of the base. Referring to Fig. 176,

let g = the distance of centre of gravity of area from the lighter load, P_1. Then

$$g = \frac{P_2 l}{P_1 + P_2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

This distance g may also be expressed in terms of l, w_1, and w_2, as follows: If the trapezoid is divided into two triangles, as in Fig. 177, the area of one triangle is $\frac{w_1 l}{2}$, and of the other

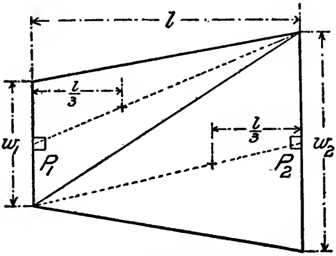

FIG. 177.—Diagram of Combined Footing, Two Unequally Loaded Columns, Trapezoidal Area.

$\frac{w_2 l}{2}$. The centre of gravity of each triangle lies on a line drawn from the centre point of the base to the opposite angle, and its distance from either base equals $\frac{l}{3}$. Then, taking moments of the triangle areas about the shorter side, w_1, and dividing by the entire area, will equal the distance g, or

$$g = \frac{\left(\frac{w_1 l}{2} \times \frac{l}{3}\right) + \left(\frac{w_2 l}{2} \times \frac{2l}{3}\right)}{\frac{w_1 + w_2}{2} \times l} = \frac{l}{3} \times \frac{w_1 + 2w_2}{w_1 + w_2}. \quad (6)$$

The area of the trapezoid

$$= A = \frac{w_1 + w_2}{2} l, \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

and the value of the second term of this equation can be obtained by dividing the sum of the two loads, P_1, and P_2, by the allowable pressure per square foot on the soil. The distance centre to centre of columns, l, is also known, so that we have the two equations (6) and (7) containing but two unknown quantities, w_1 and w_2. Solving for these we have

$$w_1 = \frac{2A}{l^2}(2l - 3g). \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

and

$$w_2 = \frac{2A}{l^2}(3g - l). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Substituting the value of g in these equations as found previously in (5), they become

$$w_1 = \frac{2A(2P_1 - P_2)}{l(P_1 + P_2)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

and

$$w_2 = \frac{2A(2P_2 - P_1)}{l(P_1 + P_2)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

To obtain the point of no shear, consider Fig. 178, with

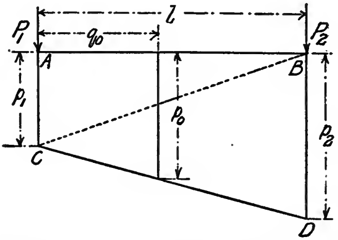

FIG. 178.—Diagram of Unit Pressures for Footing, as in Fig. 177.

the varying unit pressures on the base. Let p_2 represent the maximum unit pressure under the heavier column, and p_1 the

minimum unit pressure under the lighter column. If p denotes the allowable pressure per square foot on the soil, then

$$p_1 = pw_1, \quad \text{and} \quad p_2 = pw_2.$$

The pressure due to the load P_1 will vary from p_1 beneath the load, to 0 at the other extreme end B, and these varying pressures may be represented by vertical ordinates between the base AB and the line CB. In like manner, the varying pressures due to the column load P_2 may be represented by vertical ordinates between the lines CB and BD. The resultant unit pressure due to *both* column loads will be represented by vertical ordinates between the base AB and CD.

At the point of no shear, the ordinate at that point will equal

$$p_0 = \sqrt{\frac{2P_1}{l}(p_2 - p_1) + p_1^2}.^*$$

If p_0 is distant q_0 from the column load P_1, then, from the similarity of triangles,

$$q_0 = \frac{p_0 - p_1}{p_2 - p_1} \times l.$$

Taking moments, then, to the left of this point, and remembering that $pq_0 = P_1$, we have

$$M = P_1 q_0 - P_1(q_0 - g_1) = P_1 g_1, \quad . \quad . \quad (12)$$

where g_1 is the distance from the load P_1 to the centre of gravity of the trapezoid included between p_1 and p_0.

Changing the notation in equation (6) to suit this trapezoid, we have

$$g_1 = \frac{q_0}{3} \times \frac{p_1 + 2p_0}{p_1 + p_0},$$

* For the proof of this equation, see "Stresses in Framed Structures," p. 597, Prof. Du Bois, whose method of calculation of this case is here followed.

and on substituting this value in equation (12), M is determined, and consequently S.

As the lengths of the lower course beams vary, and as their unit pressures vary also, calculations must be made for each beam separately. If the centres of the beams are plotted on the line AB, the vertical ordinates between AB and CD will represent the unit pressures, and the total load distributed by any beam will be the product of its ordinate times the distance centre to centre of beams. The load so found should be substituted in equation (1).

Limitation.—If $P_2 = 2P_1$, then w_1 in equation (10) reduces to 0, and the trapezoid becomes a triangle. Hence if either column load is less than one-half the other, this method is not applicable.

Three Unequally Loaded Columns, Area Rectangular.—Considering Fig. 174, the line of flexure of the 15-in. I-beams

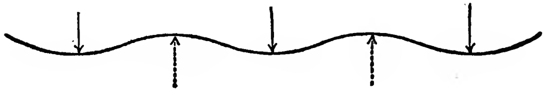

Fig. 179.—Line of Flexure for Continuous Girder.

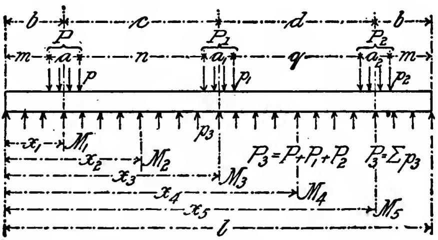

Fig. 180.—Diagram of Combined Footing, Three Unequally Loaded Columns, Area Rectangular.

will be as in Fig. 179. To find the maximum bending moment on these beams we must compute the various bending

moments and compare. The bending moment will be maximum when the shear $= 0$. In this case there are five such sections, as shown by the line of flexure; hence we must compute the moment at each point to find the greatest. The moments under the columns will be positive, causing convexity downward, while the moments between the columns are negative, causing convexity upward. Fig. 180 may then be used.

To find the distance of the centre of gravity of the loads from the left end we have

$$x = \frac{Pb + P_1(b + c) + P_2(b + c + d)}{P + P_1 + P_2}.$$

Let $\frac{P}{a} = p$, $\frac{P_1}{a} = p_1$, $\frac{P_2}{a} = p_2$, and $\frac{P + P_1 + P_2}{l} = p_3$.

The distances from the left end of the beams to the points where $S = 0$, or the distances x_1, x_2, x_3, x_4, and x_5, are then found to be as follows:

$$x_1 p_3 = (x_1 - m)p, \quad \text{or} \quad x_1 = \frac{mp}{p - p_3};$$

$$x_2 p_3 = P, \quad \text{or} \quad x_2 = \frac{P}{p_3};$$

$$x_3 p_3 = P + [x_3 - (m + a + n)]p_1, \quad \text{or} \quad x_3 = \frac{(m+a+n)p_1 - P}{p_1 - p_3};$$

$$x_4 p_3 = P + P_1, \quad \text{or} \quad x_4 = \frac{P + P_1}{p_3};$$

$$x_5 p_3 = P + P_1 + [x_5 - (m + a + n + a_1 + q)]p_2,$$

$$\text{or} \quad x_5 = \frac{(m + a + n + a_1 + q)p_2 - P - P_1}{p_2 - p_3}.$$

The bending moments at these points are readily found by taking the moments of the external forces on one side of the point in question; thus M_1 at the first point (remembering that $M = \frac{Wl}{2}$ for a uniformly loaded cantilever) is

$$M_1 = \frac{p_3 x_1^2}{2} - \frac{p(x_1 - m)^2}{2};$$

$$M_2 = Pb - P\frac{x_2}{2} = P\left(b - \frac{x_2}{2}\right);$$

$$M_3 = \frac{p_3 x_3^2}{2} - P(x_3 - b) - \frac{p_1(x_3 - m - a - n)^2}{2};$$

$$M_4 = Pb + P_1(b + c) - (P + P_1)\frac{x_4}{2};$$

$$M_5 = \frac{p_3 x_5^2}{2} - P(x_5 - b) - P_1(x_5 - b - c)$$
$$- \frac{p_2(x_5 - m - a - n - a_1 - q)^2}{2}.$$

In general cases M_2 and M_4 will be small except where the columns are very far apart, and the maximum bending moment will be at either M_1, M_3, or M_5, according to which column is the heaviest. If the cast bases are strong enough to carry the superimposed loads on their perimeters, and the long beams form the top course, the values of M_1, M_3, and M_5 will be reduced. M_2 and M_4 would not, however, be altered.

Sufficient deflection could hardly take place to increase materially the reaction under the central column, if figured as a continuous girder; but if so calculated, the clay reaction would be of a varying intensity, as in Fig. 181. Thus, from

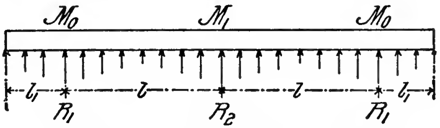

FIG. 181.—Diagram of Unit Pressures for Footing as in Fig. 180.

Clapyron's formula, we have

$$M_0 + 4M_1 + M_0 = \frac{pl^2}{2}$$

for a continuous girder of two equal spans, l. But in the case assumed

$$M_0 = \frac{-pl_1^2}{2} \text{ and } M_1 = -\frac{1}{8}pl^2 + \frac{M_0}{2}, \text{ or } M_1 = -\frac{1}{8}pl^2 - \frac{pl_1^2}{4}.$$

Taking now the shears S_1 and S_2, on the left and right respectively, of the reaction R_1, and remembering that $S_1 + S_2 = R_1$, we have

$$S_2 = \frac{M_1 - M_0}{l} + \frac{pl}{2} \quad \text{and} \quad S_1 = pl_1.$$

Then

$$R_1 = \frac{-\frac{1}{8}pl^2 - \frac{pl_1^2}{4} + \frac{pl_1^2}{2}}{l} + \frac{pl}{2} + pl_1$$

$$= \frac{1}{8}pl - \frac{pl_1}{4l} + \frac{pl_1^2}{2l} + \frac{pl}{2} + pl_1 = \frac{3}{8}pl + \frac{1}{4}\frac{pl_1^2}{l} + pl_1,$$

where $\frac{3}{8}pl$ is the reaction due to the loads on the two spans l, the same as in the regular formula for two spans, and $p_1 l$ is the reaction due to the cantilever load, while $\frac{1}{4}\frac{pl_1^2}{l}$ is the effect due to the use of the beam as a continuous girder.

Also,

$$R_2 = \frac{5}{4}pl - \frac{1}{2}\frac{pl_1^2}{l}.$$

These reactions show a varying tendency in the unit pressure on the clay, as in Fig. 181.

In the first example we made the assumption that the reaction from the clay was uniform per foot of length of the footing. According to the law of the continuous girder this would not be true, as we have seen; but when we consider that the beams are generally of sufficient depth to prevent any appreciable deflection, and that the unifying tendencies of the concrete cause the footing to act more or less as a whole, the assumption is undoubtedly justifiable.

Continuous Grillage.—By continuous grillage is meant the covering of the entire lot area with a platform of steel beams and concrete, upon which the individual footings of the columns rest. This method has been employed in several cases of high-

building construction, the idea being either to increase the area over which the structure is supported, thus reducing the unit of pressure on the soil, or to provide a rigid layer or distributing area which shall take up the strains due to any tendency toward unequal settlement, thus insuring a uniform settlement of the whole, rather than individual settlements of the separate concentrated loads.

In the St. Paul Building, the use of a uniform layer of concrete over the entire lot area has been previously referred to. This would hardly be considered as an example of continuous grillage, as the concrete layer, 12 ins. thick, was not strengthened by any steel members. The concrete was rather applied as a protective layer over a wet, sandy soil, and individual footings were used upon this, as though upon the natural surface. The unit pressure was 6,000 lbs. per sq. ft., and no appreciable settlement has been noticed.

The nineteen-story Spreckels Building in San Francisco (Reid Bros., architects) is supported upon a continuous grillage as shown by a quarter plan of the footings in Fig. 182.

"Although the main building is but 75 ft. square, the excavation, which extends underneath the adjacent sidewalks, attains dimensions of about 98 ft. by 102 ft., and was carried to a depth of 25 ft. below the street-level, where a concrete platform, 96 × 100 ft. in size and 2 ft. thick, was built over the entire surface of the dense wet sand encountered. Upon this platform was set a layer of fifty-eight 15-in. I-beams, each composed of three or four sections web- and flange-spliced to make a continuous beam 96 ft. long. Concrete was filled in level with the top flanges of these beams, and another tier of sixty-three 15-in. I-beams, similarly spliced to a length of 91.5 ft., was placed about equidistant on top of them and at right angles to their direction. More concrete was then filled and rammed to the top of their upper flanges, making virtually a solid mass of concrete, 54 ins. deep, strengthened by the

intersecting grillages. There was thus formed a platform concentric and parallel with the walls of the building, and projecting beyond them about 9 ft. on each side, so as to give an extended area for the footing upon which the weight is aimed to be uniformly distributed, 70 per cent. greater than the actual floor area of the building. This, it is planned, will

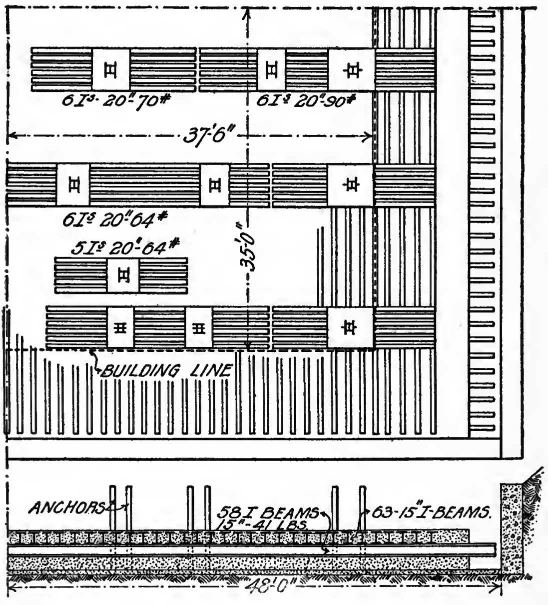

FIG. 182.—Continuous Grillage, Spreckels Building, San Francisco.

bring the unit pressure down to 4,500 lbs. per sq. ft. on the earth surface, and insure absolute continuity and uniformity in the foundation, that it may have sufficient rigidity to take up all strain and insure regular and uniform settlement, if any should occur. On top of the concrete footing surface are

placed twenty-eight sets of distributing girders, each formed of five or six parallel 20-in. rolled steel beams bolted together and supporting one or two of the forty main columns comprised in the framework of the building. Each of these columns is securely anchored by bars passing through its pedestal, and secured by keys through the webs of the lower tier grillage." *

Fibre Stresses for Foundation Beams. — The present Chicago building law specifies that if concrete is reinforced "by iron or steel beams or rails, the loads and offsets in the same must be so adjusted that the fibre strains upon the metal, if iron, shall not exceed 12,000 lbs. per sq. in., or, if steel, that the fibre strains shall not exceed 16,000 lbs. per sq. in." The same extreme fibre stresses are specified for all structural iron or steelwork.

The fibre stresses called for by the New York law are identical with the Chicago requirements.

As high as 20,000 lbs. have been used for fibre stress in steel beams and iron rails. In the Old Colony Building the steel beams in the foundations are strained to a fibre strain of 14,000 lbs. under the dead weight of the building alone, while the maximum dead- plus live-loads induce an extreme fibre strain of 21,000 lbs. per sq. in. The Carnegie strike at the time of building precluded the possibility of obtaining heavier beams than 15-in. 90-lb. I-beams, so the strain was allowed under the press of circumstances.

Steel Foundations, Painting of.—Protection from rust by means of paint, asphaltum, concrete, or "by such materials and in such manner as may be approved by the commissioner of buildings," is specified by the New York building law for metal incorporated in, or forming part of, foundation construction.

The Chicago law does not require the painting of metal-

* See the *Engineering Record*, April 9, 1898.

work embedded in concrete, thus recognizing the fact that concrete is, in itself, a better preservative than paint. Beams or rails must be entirely enveloped in concrete, the mass to be free from cavities, with all exposed surfaces coated with cement mortar at least 1 in. thick.

Pile Foundations.—The question of pile foundations *vs.* grillage methods to secure adequate support for a building is a matter of considerable difference of opinion among many architects and engineers. There are those who consider spread foundations entirely reliable, and who show their faith by using this type; while others, who question the advisability of using surface foundations for important structures, advocate piling to hard-pan. Each type undoubtedly has its favorable conditions and limitations, and the question would therefore seem to be to define such conditions of use.

First, as to the nature of the soil to be builded on, the successful use of spread foundations requires a uniform material: "uniform in character, in compressibility, in softness and in depth." Without any and all of these characteristics, the material is not adapted to the uniform settlement which must accompany grillage design. These conditions are fully met in such subsoil as is encountered in Chicago, where tests and repeated trial have shown that practically uniform settlement may be attained without resorting to the necessity for deep foundations. Very few office buildings in Chicago have been built on any other than grillage footings, except the heavy public buildings or warehouses, grain elevators, etc., along the river fronts or near Lake Michigan.

When, however, considerable variation occurs in the character of the material underlying the site, as is especially true in the lower portion of New York City, or where the substrata are of a yielding or quicksand nature, the method of spread foundations must be used with great caution. Even assuming that there is never any question as to the possible outflow of

such unstable material, as might result from relief caused by future building operations, the possibility and indeed probability of unequal settlement would require the use of piles unless the importance of the work would warrant the still greater expense and security of pneumatic foundations. Grillage foundations have been used for several very high buildings in lower New York, notably in the St. Paul Building before referred to, but in all such cases the character of the ground has been found to be very uniform and stable. By far the larger number of New York's important structures are founded either on piles, or on caissons to bed-rock.

"The first method that naturally comes to mind for providing a better foundation than can be done by simply spreading the bearings on the earth at customary depths, is that of driving piles; and where there is reasonable certainty that these will always remain wholly submerged, this is generally the best possible foundation, considering its cost, for buildings of considerable but not of the greatest weight." * But unless the driving of piles can be accomplished without injury to adjacent buildings, and without question as to the permanency of the piles themselves, the use of piling in preference to grillage may be very questionable. Their use will avoid danger through possible excavations in adjoining lots, and greater loads may generally be carried over a given area; but great care is necessary to see that the piles are not badly injured in driving, and that the upper portions are never exposed to alternate wet and dry conditions.

Test Loads on Piling.—The most satisfactory bearing values for piles can be obtained through the use of test loads as described in connection with the Chicago Public Library under the previous paragraph "Test loads." Patton states that experience and experiment are of most value in determin-

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* See Charles Sooysmith in Trans. Am. Soc. C. E., vol. xxxv.

ing the bearing-power of piles, and even with experience, experiment is much the safer rule for any other than very well-known conditions.

Test loads for piling may be obtained by driving a cluster of piles at the required site, as nearly under the actual conditions to be fulfilled in the completed structure as may be possible. They are loaded as for the Chicago Public Library test, and settlements noted. For working values, a factor of safety of from 2 to 4 is used, depending upon the thoroughness of the test, the number of tests, and the character of the building. If not driven closer than 30 ins. centres, a cluster of piles will usually bear a greater load than the summation of the loads determined for individual piles. This is due to the consolidation of the soil around each pile, thus giving more and more resistance to the remaining piles as driven. This increase, however, in the sustaining power due to compacting the earth is limited in extreme cases, as will be pointed out under the heading on pile formulæ.

Test loads should not be applied to piles until twenty-four hours or more after they are driven, in order to permit the filling in or compacting of the soil around the shaft.

Formulæ for Bearing-power of Piles.—When not specified by building law, or fixed by test loads, the safe bearing values of piles must be determined by formulæ. Of these, a great number has been devised by different authorities, and Mr. J. Foster Crowell * shows that "fourteen different values for the extreme sustaining power of the same pile, driven under precisely similar conditions, range, in a typical case, all the way from 96,000 to 600,000 lbs."

A great deal has been written concerning pile-driving and pile formulæ, and for extended information on these subjects

* See "Uniform Practice in Pile-driving," J. Foster Crowell, Trans. Am. Soc. C. E., vol. xxvii.

reference may be made to Prof. Baker's "Treatise on Masonry Construction," Patton's "Practical Treatise on Foundations," the paper by Mr. Crowell, before referred to, and other articles and books on the same subject. Two formulæ only will here be given, as constituting about the most satisfactory ones which have yet been employed.

The formula originally proposed by Mr. A. M. Wellington, and since called the "Engineering News" formula, is considered by many as more reliable and decidedly more convenient than most others of any extended use. This is of the form

$$P = \frac{fwh}{s + c},$$

where P = safe bearing resistance of pile;

f = factor, varying from 12 to 1, and recommended to be taken at 2, thus giving a factor of safety of 6;

w = weight of the hammer, in pounds;

h = height of hammer fall, in feet;

s = penetration, in inches, under the last blow of hammer;

c = constant to provide for the increased resistance to moving at moment of impact, taken equal to 1.

The following table gives the safe loads in tons, according to the above formula, for piles driven with a 1-ton hammer. For a hammer of different weight, multiply the safe load given in table by the weight of the hammer in tons.

A complete discussion as to the merits of the above formula may be found in the *Engineering News*, vol. xxix. No. 8, and from the views expressed on this subject by many well qualified to judge, it will be seen that the varying conditions of pile-driving, including the great range of material to be penetrated, the loss of energy due to broomed heads, and many other conditions of a practical nature, make it impossible to even

approximately fix the ultimate bearing-power of piles in terms of the weight and fall of a hammer, and the attendant penetration.

Last Penetration of Pile, in Inches.	Height of Fall of Hammer, in Feet.												
	3	4	5	6	8	10	12	14	16	18	20	25	30
0.25	4.8	6.4	8.1	9.7	12.9	16.1	19.4	22.5	25.8	29.1	32.3		
0.50	4.0	5.3	6.7	8.0	10.7	13.3	16.1	18.7	21.3	24.0	26.6	33.3	
0.75	3.4	4.6	5.7	6.9	9.2	11.5	13.8	16.1	18.4	20.7	23.0	28.8	34.5
1.00	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	25.0	30.0
1.25		3.6	4.5	5.4	7.1	8.9	10.7	12.5	14.3	16.1	17.9	22.3	26.7
1.50		3.2	4.0	4.8	6.4	8.0	9.6	11.2	12.8	14.4	16.0	20.0	24.0
1.75			3.6	4.4	5.8	7.3	8.8	10.2	11.7	13.1	14.6	18.2	21.9
2.00			3.3	4.0	5.3	6.7	8.0	9.3	10.7	12.0	13.3	16.7	20.0
2.50				3.4	4.6	5.7	6.9	8.0	9.1	10.3	11.4	14.3	17.1
3.00				3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	12.5	15.0
3.50					3.6	4.4	5.3	6.2	7.1	8.0	8.9	11.1	13.3
4.00					3.2	4.0	4.8	5.6	6.4	7.2	8.0	10.0	12.0
5.00						3.3	4.0	4.7	5.3	6.0	6.7	8.3	10.0
6.00							3.4	4.0	4.6	5.1	5.7	7.1	8.6

Prof. Patton * states that "after a period of rest it is evident that piles support their loads by the upward pressure at the point of the pile, and by the frictional resistance on the surface of the pile in contact with the soil," and he therefore considers that both the best and simplest way of determining the safe bearing-power of a pile is in terms of the bearing-power of the point and the frictional resistance against the surface of the pile, or

$$P = p + fs,$$

where P = safe bearing-power of pile;

p = safe resistance to settling, determined by the bearing-power of the soil;

f = factor depending upon the frictional resistance of the soil upon the surface of the pile;

s = number of square feet of pile surface in contact with the soil.

* See "A Practical Treatise on Foundations," p. 220.

For value of p, Prof. Patton gives from 5,000 to 6,000 lbs. per sq. ft. for safe load on sand, gravel, and clay, while in silt the value would be 0.

For values of f, take

100 lbs. per sq. ft. for the softest semi-fluid soils;
200 lbs. per sq. ft. for compact silt and clay;
300 to 500 lbs. per sq. ft. for mixed earths with considerable grit, and
400 to 600 lbs. per sq. ft. for compact sand or sand and gravel.

When piles are driven by water-jet, instead of by hammer blows, such formulæ as the *Engineering News* form could not apply, and it would be necessary to use some such form as Patton's, or else to determine the bearing-power by test loads. Prof. Patton recommends the use of his formula, as above, but supplemented by tests where possible.

In determining the resultant bearing-power of a large number of piles driven close together, Mr. Sooysmith has pointed out * that the value of the total number may be only the safe bearing-power of the underlying stratum supporting the pile-points. For, while a single pile or a few piles may rely on both the resistance at the point and the friction upon the sides, many piles driven closely together and to a material at all yielding may be considered as simply replacing compressible material and as transferring the load to the layer receiving the points. The bearing-power of the foundation then becomes the safe bearing-power of the stratum below the piles, plus the frictional resistance of the side walls or outer surfaces of the site, or of the mass filled with piles. It is, therefore, quite possible to overload the substratum by driving the piles too close together.

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* See Trans. Am. Soc. C. E., vol. xxxv. p. 464.

Specifications for Piles.—The specifications for piles and pile platforms in the Chicago Post-office and Government Building were as follows:

Piles.—All piles are to be of the same kind of wood, but may be of any one of the following: Hard, yellow, first-growth, untapped, Southern pine, or oak, or Norway pine. They must be not less than 10 ins. diameter at the small end and not less than 16 ins. or more than 23 ins. at the butt. Each pile must be sound throughout, of natural growth, reasonably straight and true along its entire length, properly trimmed, and the small ends sawed off to a plane normal to the axis of the pile.

The piles are to be driven with a steam hammer, the machine to be placed in the trench or on a level with the general excavation, as the contractor may choose. Should the latter method be adopted, the guides must be lengthened to reach within 4 ft. of the bottom of trench or pit, and if a follower is used the head of the pile shall be properly protected from injury by a suitable iron ring. The heads of all piles must be sawed off accurately on a horizontal plane true to the required level. All piles must be driven until they reach and fetch up hard on the very hard-pan underlying the clay, and the pile shall not sink more than $\frac{1}{2}$ in. for the last six blows of a 2,000-lb. steam hammer with full force. All piles must be of sufficient length to fulfil the above conditions whatever the soil may be. The average depth of hard-pan is assumed to be 72 ft. below the inside sidewalk grade, and is to be taken as a basis of length of pile. Should any portion of the ground require piles exceeding 48 ft. in length when cut off to the required level figured on the drawings, the contractor is to receive an additional amount per foot.

Platform.—After the heads of the piles have been sawed off, the earth around them is to be thoroughly tamped, well rammed and smoothed off to the level even with the top of the piles. The piles are then to be capped by white-oak caps,

14 × 14 ins., of lengths shown on the drawings. The caps shall be fastened to the pile-heads by means of 1-in. wrought-iron drift-bolts 24 ins. long, one in each end of each cap-timber. All joints and ends of all timbers are to be sawed square, and joints properly broken so that no two, as far as practicable, will come on the same line, and all butt-joints are to come directly over centre of pile-heads or cross-caps. The caps are to project over the outer edge of the top of the outside pile. On top of these cap-timbers a platform will be laid consisting of white-oak timbers 12 × 12 ins., closely laid in random lengths, no timber being less than 12 ft. long. The outside timbers of each platform to be bolted to each cap-timber on which they rest with one 1-in. wrought-iron drift-bolt 20 ins. long.

Water-level.—Wherever piles are employed for foundations, it is obviously of the utmost importance to establish the permanent water-level, in order that the piles may be always below this line. This may be ascertained by means of test-pits, dug to below the water-level, in which the water is permitted to remain for as long a time as may be allowed by the building operations. The water-level may then be measured at stated intervals, care being taken to prevent any unusual local disturbances, such as the inflow of rain-water or water from pipes, springs, or sewers. The line finally determined on should be low enough to provide for some reasonable lowering of the observations.

The removal of a building in New York City which had been built on piles driven some ten or twelve years previously, and the seriously decayed condition of the piles has been cited by Mr. Sooysmith as showing the danger attending the use of piles when not driven below the water-line.* Mr. Sooysmith further adds that, owing to the number of springs and driven

* See Trans. Am. Soc. C. E., vol. xxxv. p. 465.

wells throughout lower New York, "the water-level at any one point may be materially lowered at any time by pumping from a driven well in the vicinity or from the constant drainage of some leaking basement or other excavation. Thus it would seem that the permanence of any given water-level in the city can rarely be relied upon."

Pile Foundations in Chicago.—Pile foundations were used in Chicago for many years previous to the introduction of the isolated pier method, and some of the oldest and heaviest buildings are founded on them; notably the grain elevators along the Chicago River, which, in spite of their constantly varying loads, have so far maintained their integrity, though few buildings could be more trying on any type of foundations.

Some years ago the use of piles in Chicago was decried in consequence of the very careless methods and designs used in the City Hall Building. And as we look back upon the results of this work, it is hardly surprising that piles should have been viewed with suspicion for some time after by those, at least, who looked no deeper than the effect, without considering the cause. In this building the piles were driven so near together that when a new one was driven its neighbor was raised up. The foundations were put in uniformly, although the weight was far from being uniform on the different piers; and even at the time the floors were placed a variation of $7\frac{1}{2}$ ins. had resulted in the settlement.

Another example of poor pile-driving at about the same time was the foundations for the Chicago water-works tower. The surface material consisted of about 17 ft. of pure lake-shore sand, and during the later blows a very heavy hammer was needed to drive a pile even $\frac{1}{4}$ in. by measurement. But the specifications as to *depth* were to be complied with rather than any regard as to resistance, and the piles were hammered and rehammered until the sand was pierced, and a drop of 11 ins. into soft material was suddenly noticed.

After these and other failures the stone and concrete foundation was used, until the introduction of the "raft" method, which was almost universally approved, and so extensively used that the pile method was for a time quite dispensed with. But in 1889 Mr. S. S. Beman revived the use of piles in the Wisconsin Central Depot, under trying circumstances. The building itself is only eight stories high, while the tower, carried on piles at 20 tons and more per pile, is 240 ft. high. There has been no appreciable unequal settlement.

Another firm advocate of the pile foundation was Mr. Felix Adler of the firm of Adler & Sullivan. The Schiller Theatre Building, by these architects, was built on piles, "as the enormous concentrations of loads, next to adjacent walls, made it seem almost impossible to use iron and concrete foundations without an expense almost prohibitive." It was therefore decided to use piles, driven 50 ft. below datum, loaded at 55 tons per pile, and cut off at datum, with oak grillage on top and a solid bed of concrete spread over the entire area.

The work in question, however, was not at all successful as regards the adjacent property, and, indeed, such damage was done by the pile-driving that suit was instituted against the owners of the Schiller Theatre by the owners of the adjacent Borden Block, as a result of damage sustained. A similar suit was brought against the proprietors of the Stock Exchange Building.

Later examples of the use of pile foundations in Chicago are described in the following paragraph, and under the heading "Combined Grillage and Piling."

Pile Foundations in Chicago Post-office and Government Building.*—The new Chicago Post-office and Government Building is a heavy masonry structure, supported on pile foundations carried down through the overlying clay generally found in that locality, to the hard-pan which lies at an average

* See *Engineering News*, vol. xxxix. No. 4.

depth of about 72 ft. below the street-grade. The borings made at this site were previously referred to under the heading "Test Borings." About 5,000 piles were used in all, these being driven in rows for the exterior walls, and in clusters of varying size for the independent piers, etc., distributed over the site. They were spaced about 3 ft. to 3 ft. 6 ins. centres, the specifications for the pile material being as given in a previous paragraph.

The details of a pier are shown in Fig. 183. The bottom

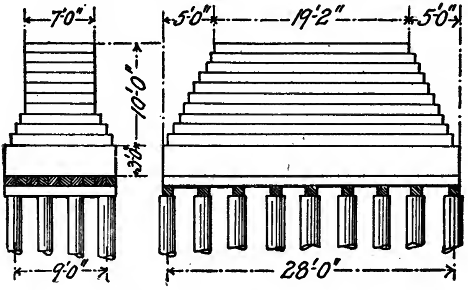

Fig. 183.—Pile Foundations in Chicago Post-office.

of the trench is about 28 ft. below the street-level, and as the piles averaged 48 ft. in net length, this made about 76 ft. from the street-grade to foot of piles. White-oak capping, 14 × 14 ins., was placed upon the pile-tops after the heads had been cut off to a uniform grade, and a close flooring of 12-in. by 12-in. white-oak timbers was then laid to support a 3-ft. bed of concrete. Upon this concrete the masonry piers were built up to the required grade, the material being limestone, laid in courses about 12 ins. thick.

The pile-driving was done with a steam pile-hammer weighing 4,400 lbs., and making 60 blows per minute.

Pile Foundations in Park Row Building.—The total weight of this building was estimated at 65,200 tons, 56,200 tons being for the weight of the structure, including wind

pressure, but exclusive of steel frame, which latter portion was estimated at 9,000 tons. The area covered is about 15,000 sq. ft., and some 3,900 foundation piles were used, thus giving about 16 tons per pile.*

Test borings indicated an underlying bed of uniform, fine wet sand, extending some 95 ft. to hard-pan or bed-rock, and this material proved so firm and solid that but few of the piles could be driven lower than 15 or 20 ft. The piles were therefore driven until the last blow showed a refusal of 1 in. fall under a 2,000-lb. hammer with a drop of 20 ft.

Under the various piers the piles were driven in rows, the

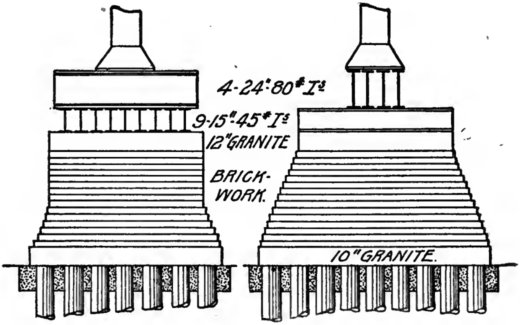

FIG. 184.—Pile Foundations in Park Row Building, New York.

piles being 18 ins. centres, in rows 24 ins. apart. After the tops had been cut off below the permanent water-level, the heads were surrounded to a depth of 16 ins. with a solid mass of concrete, composed of 1 part sand, 2 parts Portland cement, and 5 parts 2½-in. stone. A 10-in. granite capping course was then laid, upon which brick piers were built, loaded to 15 tons per sq. ft., and a 12-in. granite course was last applied to receive the grillage beams. (See Fig. 184.)

The steel grillage beams were grouted in a ½-in. bed of Portland cement mortar, and where irregularities existed

* For a detailed description of this building, see the *Engineering Record*. vol. xxxviii. No. 7.

between the beams and the granite capping of more than $\frac{1}{2}$ in., thin flat bars of steel, bedded in grout, were employed as packing.

Where two or more columns were combined as one pier, heavy box girders were used to distribute the loads.

These foundations were executed with considerable difficulty, as the walls and foundations of the adjoining buildings were not suitable to resist the vibrations caused by pile-driving. Underpinning by means of needle-beams and pipe supports was therefore rendered necessary while the adjacent foundations were removed and replaced by new brick walls and footings, carried down somewhat below the level of the new excavation.

Combined Grillage and Piling.—For the purpose of compressing the clay and thereby permitting a greater bearing unit per square foot, piles have been used in combination with ordinary grillage foundations, as in the case of the Fisher Building, Chicago, 1896. In this instance, the piles were disregarded as to direct bearing capacity, and the footings were designed as purely spread foundations.

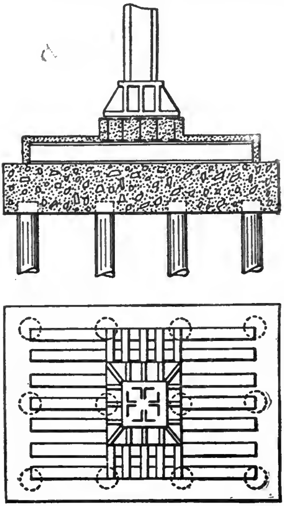

FIG. 185.—Pile Foundation in Fisher Building, Chicago.

On account of there being no party-wall contract, and also on account of the high resultant pressures per square foot for ordinary spread footings along the party-line, Mr. Shankland decided to drive short piles into the clay, thereby compressing the material and making it of the same condition before the building was commenced as ordinarily

obtains after the erection of a heavy building upon it.* Twenty-five-foot piles were therefore driven about 3 ft. centres under the footings, and it required from four to eight blows of a 2,500-lb. hammer, falling 20 to 24 ft., to drive the piles the last foot. It was therefore considered perfectly safe to load the piles to 25 tons each, or rather, as the piles were practically disregarded, to load the 9 sq. ft. of clay around each pile to nearly 6,000 lbs. per sq. ft., or almost double the usual allowance. A single column-footing for this building is shown in Fig. 185.

Another very interesting combination foundation by the same designer was utilized for an office building 40 ft. wide and 165 ft. long, where, owing to the absence of party-wall contracts, the footings were required to be entirely within the lot-lines, and shoring or underpinning would have been costly and dangerous. It was therefore decided to drive piles in the central portion of the lot, while preserving a minimum distance of 6 ft. from either side wall, as in Fig. 186. Plate girders,

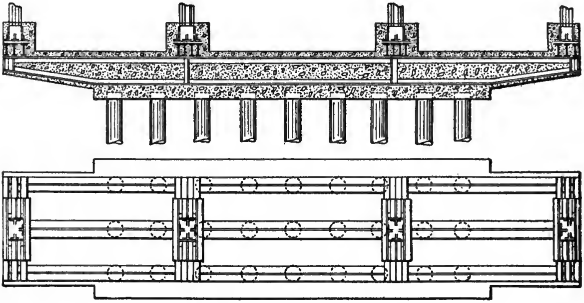

FIG. 186.—Combination Grillage and Piling.

spanning the entire lot width, were then placed over each row of piles, upon which girders the cross-beams and column-shoes

* See E. C. Shankland in Minutes of Proceedings Inst. C. E., vol. cxxviii. p. 20.

rested. Concrete was used to cover the pile-tops, and to surround the metal-work as shown in the illustration.

Use of Piles: Building Laws.—The laws of New York specify that no pile shall be loaded in excess of 20 tons. The spacing shall be not less than 20 ins. nor more than 36 ins. on centres, while the size must be not less than 5-in. end and 10-in. butt for piles 20 ft. or less in length, or 5-in. end and 20-in. butt for piles more than 20 ft. in length. For the sustaining power of piles, Mr. Wellington's formula is specified. The tops of all piles must be cut off below the lowest water-line.

The Chicago building law requires that piles be driven to rock or hard-pan bearing, the safe load to be according to approved formulæ for pile-driving, but not exceeding 25 tons per pile. A capping of oak grillage is specified, the extreme fibre stress not to exceed 1,200 lbs. per sq. in., the top of such oak grillage to be at least 1 ft. below city datum or 1 ft. below the bottom of any adjacent sewer which may be below city datum.

The Boston law does not specify any unit loads for piles, the requirements being that the piles shall be not more than 3 ft. apart on centres in the direction of the wall, "and the number, diameter, and bearing shall be sufficient to support the superstructure proposed." The walls of buildings over 70 ft. in height must rest, where possible, upon at least three rows of piles, all to be capped with block-granite levellers.

Foundations to Bed-rock.—Foundations to bed-rock have always been recognized as particularly desirable for any and all forms of heavy building construction, but open excavations (such as were secured before the introduction of modern methods) become impracticable under the present conditions obtaining in large cities, owing to the safety which must be accorded adjacent structures.

The greatly increased height and consequent weight

developed in modern buildings have required a corresponding extension or development of foundation methods, and as great security and absolute integrity have been demanded of the designer, even when building upon soft or treacherous soils, the necessity for reaching bed-rock has had to be met, and often under conditions so difficult that the proceeding would have been impossible under former methods.

When bed-rock is to be found at no great depth, there can be little question as to the desirability of securing rock foundation for any structure of importance, provided the cost of such foundation be not disproportionately large. The added security would warrant a reasonable increase in cost, and this added outlay becomes a smaller percentage on the entire work as the total cost and importance of the structure is increased.

If rock bottom is at great depth, and the soil presents uniform conditions suitable for grillage design, there can be no good reason for incurring the increased expense of caissons; nor, if the driving of piles seems expedient, should caissons be preferred at largely added cost. But if bed-rock is fairly accessible, or if at considerable depth and overlaid with quicksand or soil containing water-bearing strata, recourse must be had to some form of deep-foundation design. This is now accomplished by means of caissons, of which two types have been extensively used—hydraulic caissons or open cylinders, and pneumatic caissons.

Open Cylinders or Hydraulic Caissons.—Open cylinders to bed-rock are only applicable where sand or earthy soils free from bowlders or other obstructions are to be penetrated, and where the extensive pumping and jetting of water made necessary by this process will not cause undermining tendencies in soft or unstable soil under adjoining buildings.

This method consists of sinking steel or wood cylinders, either circular or rectangular in cross-section, from the surface to the rock bottom. The cylinders are usually made of $\frac{3}{8}$-in.

steel plates, in sections about 3 ft. long and from 6 to 10 ft. in diameter, according to the bearing area required in the pier. For moderate depths the cylinders are often delivered at the site completely riveted up, but for any great depths the sections are field riveted as fast as the shell is sunk. The connections between the several sections are made by means of lap-joints, with $\frac{3}{4}$-in. field rivets, pitched about 5 ins. Wooden cylinders are also employed, as in the new Stock Exchange Building in New York.

The bottom edge of the cylinder is fitted with a cast-iron or steel cutting-edge, which is provided with nozzle attachments, so that water-jets at about 100 lbs. pressure may be delivered through orifices in the cutting-edge. The first section is started in a pit dug to the water-line, and then by loading the tops of the cylinder, and by starting the water pressure through the cutting-edge, the earth is scoured out below the shell and so softened that the applied load gradually sinks the cylinder through the soft material to a rock bearing, but still leaves a vertical earth core within. The cylinder is then excavated, and either filled with concrete, or a bed of concrete some 4 or 6 ft. thick is placed at the bottom, upon which brick piers are started of the full size and height of the pier. Grillage beams are then applied to receive the column-stands, as illustrated in Fig. 187.

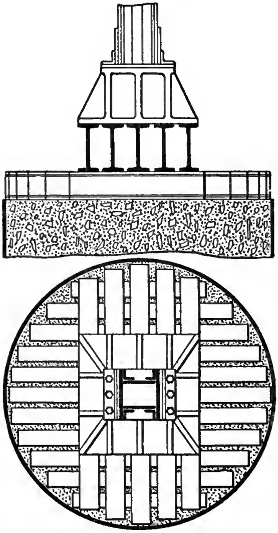

FIG. 187.—Open Cylinder Foundation.

Pneumatic Caissons, Use of.—This method of securing

deep foundations in water, quicksand, or unstable soils, has been very extensively developed in bridge building, and by far the larger number of masonry piers for important railroad bridges has been founded on caissons driven to bed-rock or hard-pan by the pneumatic process. In general principles and even in details the pneumatic caissons employed in building construction differ but slightly from those used in bridge work, and for extended information on this subject reference may be made to Prof. Baker's "Treatise on Masonry Construction," to Prof. Patton's work on "Foundations," or to any of the detailed reports submitted or published by the chief engineers of prominent bridge work.

Pneumatic caissons have been and are now being employed in many of the most important high buildings, especially in New York City. The process has been found most reliable under the severest conditions. The advantages secured by this process are, first: excavations may be carried on under a sufficient air pressure to insure the holding back of any inflowing outside and unstable material; second, obstructions encountered in sinking the piers, such as logs or bowlders, may be removed; third, the rock bottom may be examined and, if necessary, levelled off or stepped to secure a firm bearing; fourth, the piers can be built while the caissons are being sunk, so that the piers are completed as soon as the bed-rock is reached.

Regarding the proportional cost of this type of foundations, Mr. Charles Sooysmith states as follows: * "The pneumatic process is the one safe and sure method for deep excavations by which all dangers of quicksand or other difficulties can, with certainty, be quickly overcome and a perfect foundation constructed; and this, too, at a cost, where the conditions are determined, which can generally be estimated with compara-

* See "Concerning Foundations for Heavy Buildings in New York City," Trans. Am. Soc. C. E., vol. xxxv. p. 468.

tive certainty." . . . "It is probable that a sum not exceeding 3 or 4 per cent. of the cost of the entire building, added to what the cheapest possible shallow foundation would cost for one of the very high buildings, would cover the extra cost of carrying its foundations to the solid rock, when this is within 70 or 80 ft. of the surface. In many cases this extra cost would be more than offset by the value of the additional story or stories that could be provided beneath the surface."

Pneumatic Caissons, Design of.—A pneumatic caisson consists of a circular or rectangular box of wood or steel, with flat top and vertical sides, but open at the bottom. A cross-section of an ordinary form is shown in Fig. 188. The top or roof is sometimes constructed of solid layers or courses of timbers, and sometimes by alternate courses with spaces between the timbers. In the latter form, the voids are filled with concrete. The side walls are usually made solid, of about 12-in. by 12-in. timbers, while the lower edges or "cutting-edges" are provided with a steel plate or shoe of some form to act as a cutting- or penetrating-edge into the underlying material. The whole construction is designed to be air-tight.

The interior or working chamber is connected with the exterior by means of "air-shafts," which consist of vertical circular shafts extending through the roof and up through the pier, these being extended by means of successive sections, as the chamber descends. Two or more of these shafts are usually provided for the use of the workmen and for the carrying of the earth or other excavated material from the inside to the surface for carting away.

Each air-shaft is provided at its upper end with an "air-lock," consisting of a small steel chamber which has two doors—one connecting with the vertical shaft leading to the working-chamber, and the other connecting to the outside air. As the inside chamber is filled with compressed air, the two doors to the air-lock may never be opened at the same time—

otherwise the compressed air would escape, and the working-chamber would quickly fill with water, if below the water-line. For the passage of materials, the air-locks are operated as quickly as possible, both to save time, and to cause the least possible escape of compressed air; but, for the passage of work-

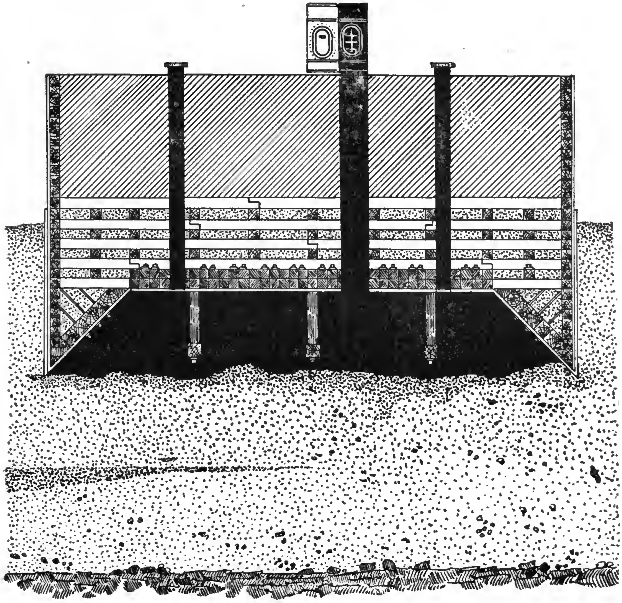

FIG. 188.—Section through Pneumatic Caisson.

men, the transition from one atmosphere to the other must be made more gradually, in order that injury to the inmates may not be caused by the sudden increase or diminution of pressure.

Caissons may be built in position or delivered at the site ready for use, according to the size and facilities for handling. When exactly located upon the surface material where the pier

is to be sunk, the men in the working-chamber start excavating the underlying soil, and undermining the cutting-edges, so that the caisson gradually sinks under the superimposed load. This may be started with open air-shafts, but as soon as the water-level is reached, and water becomes troublesome in the working-chamber, the locks must be closed and air pressure turned on of a sufficient pressure to keep the chamber free from water. Compressed air is furnished by means of compressors located at the site. The excavated material is hoisted to the surface by means of buckets working in the material shafts, but if sand or fine soil is encountered, the material is discharged at the surface by means of the sand-pump, which consists of a vertical pipe, open at the surface, but sealed at the lower end by means of a puddle of water maintained below the level of the caisson bed. The fine material held in suspension is drawn up by the suction obtained by discharging compressed air around the discharging nozzle of the sand-pipe.

Water-tight coffer-dams are usually extended above the roofs of the caissons, so that the caissons may be sunk without necessarily waiting for the starting of the masonry piers. Time will be saved, however, if the piers are built while the caisson is being sunk, and the added weight of the piers is often valuable in causing the caisson to follow the excavation. In small caissons with vertical sides, such as are often employed in building work, the friction of the sides often becomes so great that a temporary loading of pig-iron is necessary, even in addition to the masonry pier, in order to sink the caisson against the friction and the upward pressure of the compressed air.

When the caisson has reached the required level, the bed-rock is levelled or stepped off, as may be necessary, the surface is carefully cleaned, and the working-chamber and air-shafts are filled with concrete.

Caissons are now lighted by electricity and telephone communication with the surface is sometimes provided. Bowlders are removed by blasting with dynamite.

First Use of Pneumatic Caissons.—Pneumatic caissons were first employed in building construction in the Manhattan Life Insurance Building, New York City. The building proper is seventeen stories high, with a tower on top terminating in a dome. The main roof is at an elevation of 242 ft. from the sidewalk, and from sidewalk to base of flagstaff = 347 ft. 6 ins., and from base of foundations to top of dome = 408 ft. This makes the dome 61 ft. higher than the neighboring spire of "Old Trinity."

The area of the lot is, approximately, 120 ft. deep × 67 ft. frontage, or 8,000 sq. ft., which, with the estimated total weight of the building of some 30,000 tons, would give a load of 7,500 lbs. per sq. ft. of lot area.

The natural soil at the site consisted of mud and quicksand to a depth of some 54 ft., down to bed-rock. Had piles been used, as close together as the New York building law allows, or 30 ins. centre to centre over the entire area, some 1,323 piles could have been driven, with an average load of 45,300 lbs. each. This was inadmissible, as the building law limited the load per pile to 40,000 lbs. each, when driven 2 ft. 6 ins. centres.

A new departure in foundations was therefore necessary, especially as the surrounding buildings were built on the natural earth, making them particularly liable to injury in case of any increase of pressure on the soil from additional loading, or decrease in pressure through deep excavations or trenches for piles or concrete piers below the adjacent footings.

Pneumatic caissons were thus adopted, and this was the first example of the pneumatic system as applied to buildings, although the same architects (Messrs. Kimball & Thompson) had before used smaller caissons in the Fifth Avenue Theatre

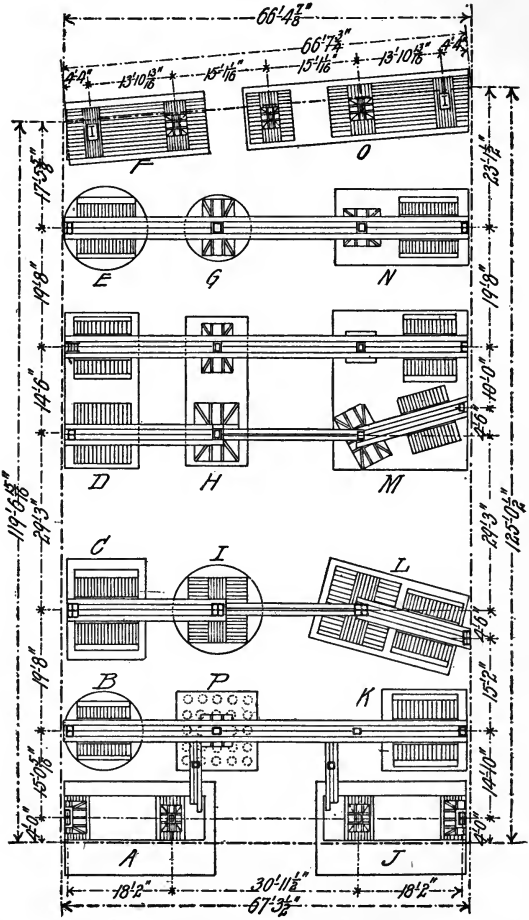

FIG. 189.—Plan of Caissons in Manhattan Life Insurance Co.'s Building, New York.

Building in New York City, but without the use of compressed air.

Fifteen caissons, varying in size from 9 ft. 9 ins. in diameter to 25 ft. square, supported the thirty-four cast-iron columns. These caissons were sunk to an average depth of about 31 ft. 6 ins. below the bottoms of the excavations at the site. After the caissons were sunk to bed-rock, the rock surface was dressed and stepped as required, and the chambers and shafts were then rammed with concrete, composed of 1 part Alsen cement, 2 parts sand, and 4 parts broken stone. The superimposed piers were built of hard-burned brick laid in cement mortar. About eight days were required to sink each caisson. The locations of the several caissons are shown in Fig. 189.

A very elaborate system of cantilever girders was used to transfer the loads on the columns in the side walls to proper

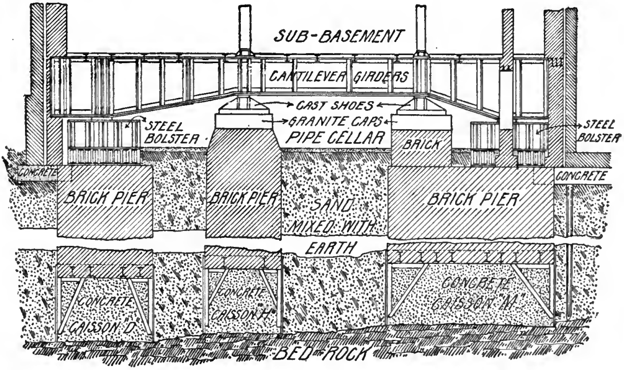

FIG. 190.—Cross-section of Caissons in Manhattan Life Insurance Co.'s Building, New York.

concentric bearings over the caisson piers. From these bearings the load was distributed over the whole masonry work by means of large steel bolsters, thus diminishing and equalizing the unit-pressure. A cross-section of the caissons and cantilever girders is shown in Fig. 190.

Pneumatic Caissons, Gillender Building.—This building, shown in Fig. 28, is 310 ft. high from the top of the grillage beams to the top of the dome. The narrow width of the

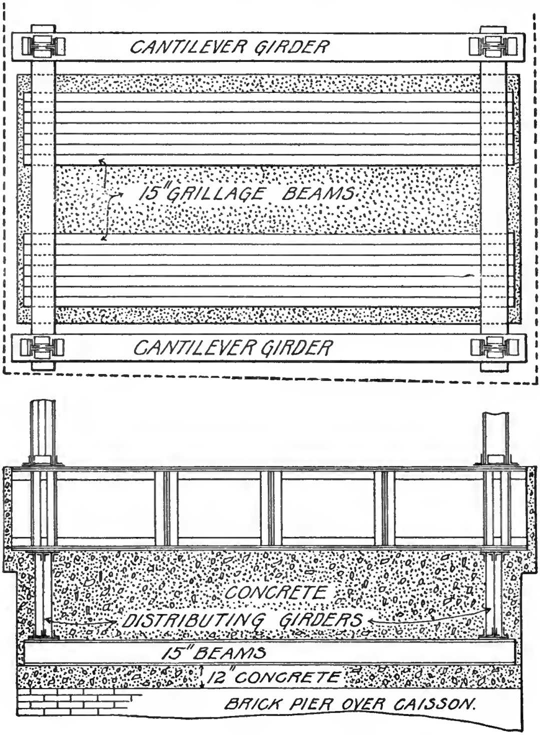

FIG. 191.—Plan of Caisson, Gillender Building.

building permitted all of the columns to be located within the exterior walls, and six columns were placed on each side, as shown by the framing plan, Fig. 60.

The foundation material consisted of fine loose wet sand, and it was found that it would be impracticable to support the

structure on any form of grillage or spread foundations, even though the entire site area (viz. 1,852 sq. ft.), were covered, as the estimated load and the pressures developed by wind

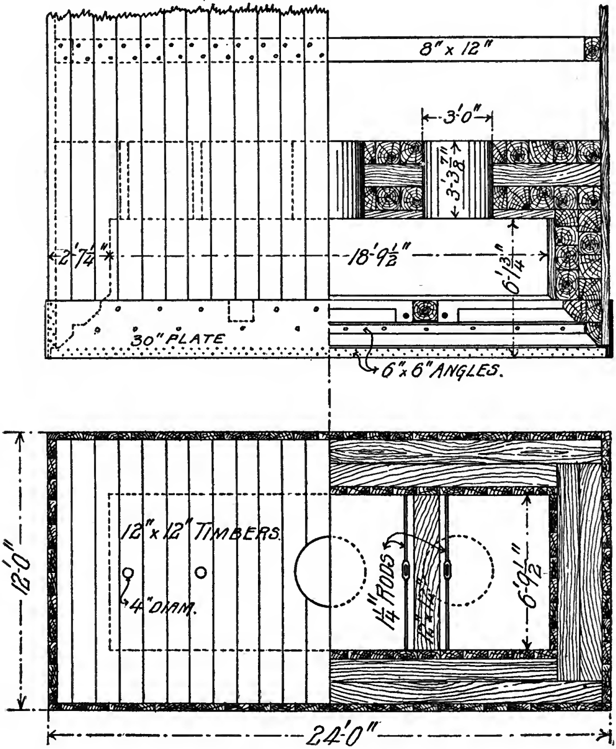

FIG. 192.—Detail of Caisson, Gillender Building.

strains would exceed the permissible bearing. Pneumatic caissons were therefore adopted, covering about three-fifths of the area of the site. Each caisson supports four columns, or two on either side of the building, as shown in Fig. 191.

These were proportioned to distribute the loads at 12 tons per sq. ft.

The general details of the caissons are shown in Fig. 192, while Fig. 193 shows a large section through a side wall and cutting-edge.

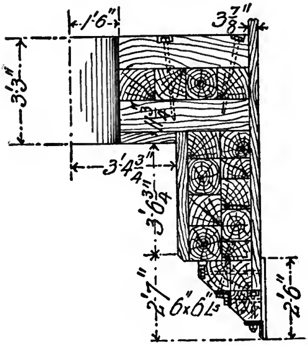

FIG. 193.—Detail of Caisson Cutting-edge, Gillender Building.

Permanent coffer-dams were extended above the tops of the caissons, thus forming vertical continuations of the caisson sides for the enclosure of the brick piers which were started upon the decks of the caisson chambers. These brick piers were about 18 ft. high. The total depth from cellar floor-line to bottom of cutting-edges was about 42 ft.

The caissons, as in Figs. 192 and 193, were built of yellow pine, with a steel plate cutting-edge as shown. The timber used was planed on all sides, and the outside planking was placed vertically to reduce the skin friction. The actual time required for sinking was seven days for the centre caisson, 15 ft. × 24 ft., and four days each for the end caissons, 12 ft. × 24 ft. each.

Over the brick piers, which were laid in Portland cement mortar, a 12-in. layer of concrete was placed to receive the grillage beams and cantilever girders. These were first painted,

then coated with coal-tar, and then surrounded by a solid mass of concrete, the minimum thickness of which was 12 ins. The interior spaces of the box girders were filled with Portland cement grout, to guard against corrosion.

Pneumatic Caissons, American Surety Building.—A framing plan of this building is shown in Fig. 61. The structure is about 85 ft. square, and twenty-one stories high, or

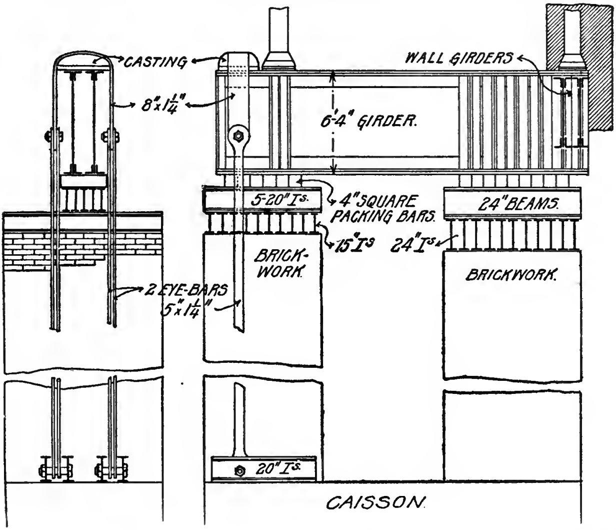

FIG. 194.—Foundation Piers, American Surety Co.'s Building, New York.

290 ft. from sidewalk curb to roof. The estimated maximum weight of the building was some 26,000 tons, and this was transmitted to bed-rock about 72 ft. below the sidewalk-grade by means of brick piers and pneumatic caissons. Thirteen steel caissons were employed, with a total distributing area of 3,575 sq. ft., the pressure per square foot thus being about 14,500 lbs. All of the caissons were rectangular, the largest being 11 ft. by 42 ft. in area, and 9 ft. high, supporting four

columns. The brick piers are about 30 ft. high, with steel grillage beams on the tops for the support of the column bases.

On two sides of the building, the wall columns are located very near the building-line, and as the caissons underneath these columns could not be extended upon the adjacent property, it became necessary to devise means for overcoming the heavy eccentric loading which would have resulted in applying the column loads direct to the caissons in the lines of the column axes. This was accomplished by connecting inner and outer piers by means of heavy box girders, which rested on grillage beams over the centres of the piers as shown in Fig. 194. The girders projected at each end beyond the grillage supports, the outer or wall ends forming cantilevers to carry the wall columns, and the interior overhanging ends forming anchorage ends, which, by anchoring down to the brick piers, served, with the interior columns applied centrally over the inner piers, to counterbalance the wall column loads. The walls were carried on plate girders running between the cantilever girders.

CHAPTER X.

SPECIFICATIONS—INSPECTION.

ADEQUATE specifications as to character of materials and workmanship, and competent inspection to provide for the enforcement of such specifications, are quite as important as intelligent and economical design. Much thought may be expended in preparing careful plans and details of the work, the execution of which may largely be rendered nugatory through loosely drawn specifications or through the lack of enforcement of carefully specified requirements.

As the steel frame for any building constitutes by far the most important portion of the structure, the specifications and provisions for the inspection of this part of the building should be most clearly and carefully stated. Specifications in sufficient detail must also be furnished for all of the classes of work which enter into the building's construction, but as these are more architectural than engineering in character, no attempt will be made here to cover other than the steelwork.

It may be noted, however, that specifications covering fireproof floor-arches, roofs, partitions, column protections, etc., are generally totally inadequate in comparison to the great importance of these features. In the report of the board of engineers who examined the effects of fire upon the Horne buildings in Pittsburg, it was recommended that the insurance companies undertake the preparation of standard specifications governing the character, construction, and methods of applying all fireproofing materials, and requiring all owners either

to use such fireproofing materials subject to these specifications and careful inspection, or else to be subjected to higher rates of insurance,—in other words, to vary the cost of insurance according to the character of the fireproofing used. Such practice would certainly lead to a decided improvement in our fireproofing methods.

For specifications regarding masonry walls, piers, etc., reference may be made to Prof. Baker's "Treatise on Masonry Construction," or to Kidder's "Building Construction and Superintendence," vols. i. and ii. The latter treatise also includes much information concerning such specifications as Carpentry, Plastering, Painting, etc., etc. For more extended data regarding fireproof floors, roofs, column casings, and partitions, besides a consideration of the elements of general fireproof design and equipment for fire resistance, see "The Fireproofing of Steel Buildings," by the author.

Specifications for Structural Steel.—These should fully and carefully cover all of the requirements of the architect or engineer in regard to the steel framework during its manufacture, fabrication, and erection—thus embracing the questions of:

1. Quality of material.
2. Shop-work and painting.
3. Inspection.
4. Erection.

Some of the points requiring especial emphasis under these various headings will be considered before detailed specifications are quoted.

Quality of Material should include requirements covering chemical constituents, physical properties, and the general finish of the plain material.

Quality. Chemical Constituents.—The *desired results* should be clearly specified, rather than processes or details of attaining results; and it will generally be found, for any ordi-

nary work, that commercial grades of material of known uniformity, in ample and usual sections which can be purchased of several different makers and furnished in prompt deliveries, are generally preferable to special grades of material or special sections.

The fact that results have been attained, or the prevention of serious delays if they have not, should be determined by prompt testing and inspection, for which facilities should be definitely specified.

The chemical composition of the steel should not be specified, other than possibly to limit the quantity of deleterious constituents, such as phosphorus, sulphur, manganese, etc., while all other elements, except carbon, are to be entirely absent or merely traceable in quantity. "No engineer should, unless he be an expert steel-maker, attempt to specify an exact chemical formula and a corresponding physical requirement; in doing so he would probably make two requirements which could not be obtained in one piece of steel, and so subject himself to a back-down or to ridicule, or both. On the other hand, he may properly, and he should, fix a limit beyond which the hurtful elements would not be tolerated."*

All steel should be specified of open-hearth manufacture and of uniform quality. Rivets to be of "soft" steel, all other steel to be of "medium" grade, as specified under the requirements for physical tests. Chemical analyses should show not more than the following quantities of phosphorus and sulphur:

	Phosphorus.	Sulphur.
Acid steel......	.08 per cent.	.06 per cent.
Basic steel......	.06 "	.05 "

There has been much recent discussion about these limits, but the above percentages will give a thoroughly satisfactory material which will still come easily within the practice of a

* See "A Manual for Steel-users," by Wm. Metcalf, p. 157.

good rolling-mill. The elements mentioned should be determined by chemical analyses, made and furnished by the rolling-mill and checked by the inspecting engineer. Analysis should be made for each original furnace heat.

Physical Properties should cover ultimate tensile strength, elastic limit, elongation, and reduction in area, and these can best be specified by calling for certain physical tests for which the manufacturer should be required to prepare test specimens, and to furnish the use of testing-machines and the necessary labor for making tests, without additional charge.

For steel, specimens should be cut from the finished material for each original furnace heat, and for each different section of material; for wrought-iron, from each section of material and for every certain number of tons; for cast-iron, from each cupola or furnace charge, and cut from or attached to the castings to be used, or, if this is not practicable, they should be cast separately from the same pour.

Test specimens, where possible, should be of sufficient length to permit the elongation to be measured in 8 ins., but specimens representing pins, small special castings, etc., may be slotted out and turned down for 3 ins. clear, or less, and measured for elongation in 2 ins., or less, and their required elongation percentage should be more than for 8 ins. length. All turned specimens should have easy fillets. Test specimens with sharp fillets will often fail more readily at less tensile strength than specimens with easy fillets.

In specifying the physical requirements for ultimate strength in steel where two or more grades are called for, as "medium" and "soft" steel, the limits for ultimate strength *should not overlap*, otherwise the rolling-mills are very apt to attempt the furnishing of *one grade* for both requirements, and thus frequently fail to get good results within the narrow limits. The specifications must then be waived, or many apparently unreasonable condemnations made. It is better to allow

liberal limits, and to hold to them. Mr. Waddell, in his treatise "De Pontibus," recommends ultimate tensile strengths per square inch as follows:

Soft steel..........	50,000 lbs. to 60,000 lbs.
Medium steel.......	60,000 lbs. to 70,000 lbs.
High steel.........	70,000 lbs. to 80,000 lbs.

It will be noticed that none of these limits overlap.

The elastic limit is usually specified to equal at least one-half of the ultimate strength, the elongation not less than 24 per cent. or 25 per cent. in 8 ins., and the reduction in area not less than about 40 per cent.

Bending and drifting tests are confirmatory of tensile tests, and if specified, should be required to be made. Not one inspector in ten does make them. Drifting tests are best accomplished by punching a hole, as in ordinary riveted work, and increasing the size with a drift-pin. For medium steel the diameter should be increased one-half without cracks at periphery of hole or edge of piece.

Forging, annealing, or any similar treatment of test specimens should be prohibited.

General Finish of Material.—Excellence of finish in the plain material before fabrication includes surface perfection, or the exclusion of defects or unsoundness in the metal or of material with "wind," and undue variations in the cross-section or weight.

For variations in weight, the usual clause about "$2\frac{1}{2}$ per cent. variation from required weight or section being cause for rejection" is adequate, except that wide plates should be shown on plans by thickness on the edge or by weight per lineal foot. See Standard Specifications of Association of American Steel Manufacturers for allowances for overweight of wide plates rolled to gauge.

Shop-work can best be controlled by placing orders with

shops which are well equipped to perform the class of work desired, and which are not too busy with other contracts. There is much difference in the character of workmanship due to detail shop management. Poor shop-work can be discovered and prevented by inspection.

Several points worthy of especial emphasis in shop-work or shop-inspection are as follows:

Plans.—All working plans or details made by the shop should be required to bear the signature of approval of the engineer or architect of the structure. Said engineer or architect should keep an original approved blue-print on file, as tracings may be changed.

Punching.—The diameter of die should not exceed the diameter of the punch by more than $\frac{1}{16}$ in.

Assembling.—Material should be straight before laying out, and, if necessary, straightened after punching. Small shops without facilities for straightening heavy angles and shapes are at a disadvantage for heavy riveted work, as such material is always more or less distorted by punching. Members should be straight, not in wind, before riveting, and a sufficient number of temporary holding-bolts should be used.

Reaming, which is often required for important connections in building construction, as in column splices, is rarely clearly specified.

There are three kinds of reaming and two kinds of drilling. (1) Reaming may be a removal of material distressed by punching, when specifications should provide for the holes to be punched of less diameter than the finished size of hole, and reamed to full size. This is done under a drill-press on individual pieces of material and does not necessarily give holes that match or insure good riveting. (2) Reaming may be specified as "fairing" the holes, and is done by a portable reamer at assembling when the various pieces of a member are brought together. It does not necessarily remove distressed

material, but tends to improve the riveting, and this is generally done by all good shops. (3) Reaming may be specified so as to improve both material and riveting by means of strict specifications regarding the considerably smaller size of punched holes and their exact matching, or by requiring the holes to be reamed at assembling, with all pieces in position.

The drilling of pieces separately does not necessarily improve the riveting.

Reaming should either be clearly specified or not specified at all.

Riveting is generally clearly covered in standard specifications, but the calking of rivets with a chisel, or by squeezing the heads cold with a smaller die, or striking them on the sides with the machine when cold should be unquestioned cause for condemnation. These last two methods have superseded the clumsier use of a chisel, and are apt to escape an inexperienced inspector.

Painting.—In no detail of manufacture are more sins committed than in painting. Rust should not be permitted; scale should be removed; paint should be well brushed on under cover when temperature is above freezing, and on dry surface; paint should be allowed to dry between coats and before shipment, preferably for 48 hours. There should be no opportunity for water to collect or to start rust at any point. Paint should be carefully identified as the brand specified, and chemical analysis can be made with advantage. There are many cases of adulteration or substitution of paint; i.e., a substitute colored with aniline instead of red lead, with a difference of cost of about $1.00 per gallon; a similar substitution for graphite, at a difference in cost of about 50 cents. Linseed oil is rarely used as specified, and many of the substitutions for and adulterations of both paints and oils can only be discovered by analysis.

For a more extended discussion as to paints and painting, see Chapter III, pages 82 to 88, inclusive.

Erection.—It is desirable to have an inspector or superintendent at the building site, who shall be capable of supervising the erection in all its details. He must see that all pieces are erected in their proper places; that riveted or bolted connections are made as specified; that the painting is properly done; that floors are not overloaded; and, generally, that plans, specifications, and good practice are followed. A good man will also greatly assist the foreman of erection in securing the correct placing of pieces, and in intelligently directing any necessary changes or corrections.

Specifications.—The following general specifications for structural iron and steel are from the practice of Hildreth & Co., Inspecting and Consulting Engineers.

Specifications for Structural Iron and Steel.

General.—All structural iron or steel shall be the best of its kind, both as regards quality of material and manufacture, and shall strictly comply with plans as regards dimensions.

Deliveries shall be made in the order required for construction, and at the time specified in the contract. If shipment of material from the foundries or rolling-mills or finished work from the shop is not made at the time agreed upon, the architect may purchase materials in the open market at such terms and for such deliveries as in his opinion shall meet the requirements of construction, and the cost of such material so purchased shall be deducted from the amount due under the contract.

Weights.—A variation of two and one-half per cent. ($2\frac{1}{2}\%$) for steel and three per cent. (3%) for cast-iron from the

estimated weights will be allowed in the finished material. Any individual member or piece of material which weighs less than the estimated weight and this allowance, may be condemned at the discretion of the architect, and any classification of material which exceeds the estimated total weight of such class by more than the variation allowed will not be paid for.

CASTINGS.

Quality.—All castings shall be of tough gray iron, free from all shrinkage-cracks, blow-holes, cold-shuts, sand, cinder, or other defects, clean, true to pattern, and neat as to finish. Only such scrap iron as may be approved by the architect or his inspector shall be mixed with the metal used for casting. Castings shall be allowed to cool slowly in the sand to avoid shrinkage-strains.

Tests.—Two specimens, each 1 in. square, shall be cast for each furnace heat as runners on different castings or from separate parts of the pour, and shall be capable of sustaining a central load of 2,500 lbs. when set on knife-edge supports 12 ins. apart, with a deflection not less than $\frac{3}{16}$ of an inch, and when turned to a diameter of about $\frac{3}{4}$ of an inch for a distance of 4 ins. shall develop a tensile strength of at least 18,000 lbs. per square inch. A blow from a hammer upon the rectangular edge of any casting shall result in an indentation without flaking the metal. Castings shall not break when struck with a sledge.

Columns.—The thickness of any part of the shell shall not vary more than $\frac{3}{16}$ in. from any other part, nor more than $\frac{1}{16}$ in. less than the thickness specified.

Fillets.—Brackets and flanges shall be boldly filleted, and in no case with fillets of less than $\frac{1}{2}$ in. radius.

STEEL.

Quality.—All steel shall be uniform in quality, and manufactured by the open-hearth process. Chemical analyses for

each original furnace heat shall be made and furnished by the rollling-mills and checked by the inspectors.

Steel shall not contain more than .08 per cent. of phosphorus, nor .06 per cent. of sulphur.

Rivets shall be "soft" steel, and all other steel shall be of "medium" quality as specified below.

Tests.—Rolling-mills rolling the steel shall furnish two test specimens cut from finished material of each original furnace heat, to identify which all material shall be marked with the number of the original furnace heat from which it is rolled. One specimen for each heat shall be broken by tension in a testing-machine, and shall show in pounds per square inch an ultimate strength of from 60,000 to 68,000 lbs. for "medium" steel and 52,000 to 60,000 lbs. for "soft" steel; an elastic limit of at least one-half the ultimate strength; and an elongation in 8 ins. of at least 25 per cent. If the first specimen fails to fulfil the above requirements, four other specimens may be tested at the discretion of the inspector, and if two also fail, all material rolled from such furnace heat shall be condemned. The second specimen shall be tested by bending one end cold, and the other end shall be heated cherry-red and quenched in water and bent; both bends shall be 180° flat without flaw.

Finish.—Finished material shall be straight, true to section, with smooth clean surface, and free from cracks, seams, buckles, or other defects.

Inspection.—The rolling-mills shall furnish all test specimens and the use of testing-machine, together with all labor necessary for handling material for inspection, without charge. No shipment shall be made without at least two days' notice to the architect or his inspector, and in the event of shipment from mills without such notice, or without proper facilities for inspection, the cost of subsequent inspection at the shops of material so shipped shall be paid by the rolling-mills, if so required by the architect.

WORKMANSHIP.

General.—All workmanship shall be first-class in every particular, and in accordance with the best modern shop-practice.

Plans.—All working shop-plans shall conform to the plans furnished by the architect, and must bear his signature of approval before work commences. Such approval, however, shall not relieve the shop from the responsibility of correcting, without charge, any errors in not following the architect's plans, or errors of "clearance" or "connections" which can be discovered by examination.

At least two sets of working plans and two copies of order lists of material shall be furnished the architect.

Foundry-work.—All machined surfaces of castings shall be accurate and smooth. Columns shall be of exact height, with bearing surfaces at right angles to the axis of the column. Connection-holes shall be accurately spaced and drilled to exact position, if necessary to an iron template, in order to provide for tight-fitting turned bolts. The depth of bracket-webs shall be twice the horizontal projection.

Punching.—All rivet-holes shall be accurately spaced in a true line, and laid out by template. The clearance between die and punch shall not exceed $\frac{1}{32}$ in. for material $\frac{1}{2}$ in. thick, nor $\frac{3}{64}$ in. for thicker material. Holes shall be clean-cut without cracks, and burrs shall be removed by a countersinking reamer.

Built girders shall have rivet-holes punched $\frac{1}{8}$ in. small, and holes shall be reamed to full size with parts in position.

Straightening.—The material for all built members shall be straightened after punching.

Assembling.—At assembling, and before riveting, built members shall be truly straight and out of "wind," held by a sufficient number of bolts to prevent warping or bending

under handling and riveting. No drifting of holes shall be done under any circumstances in any class of work, but failure of holes to match shall be corrected by new material or by reaming, at the discretion of the architect or his representative.

Rivets, shall be of soft steel driven by machine wherever practicable. They shall completely fill the holes and be tight with neat cup-shaped heads concentric with the holes and free from cracks at edges. Rivets showing evidences of burning will be rigidly condemned. In removing defective rivets, any injury to the material will be cause for condemnation of injured parts.

Connections.—All joints shall be fully spliced.

All framed beams shall be secured in position by angle-brackets and standard connections.

Connections shall be made by rivets or turned bolts fitting tight, as shown on plans.

Any beam or girder that is longer or more than ½ in. shorter than required for its special place shall be rejected. The accurate adjustment of the lengths of framed beams shall be made by reaming connection-holes and setting out angle-brackets at their ends to correct length.

Painting.—All metal-work shall be free from dust, dirt, and scale; no painting shall be done in wet or freezing weather. Except for cast-iron, all surfaces in contact and all places inaccessible at erection shall be painted one coat of paint at assembling, and finished members shall be painted one coat before shipment. After erection, all surfaces, including cast-iron, shall be painted one thorough coat. The paint used shall be the . . . . made by . . . . , and it shall be well brushed on and worked over the entire surface.

Anchors.—All beams resting on walls are to be securely anchored by approved T anchors built into the wall.

Inspection.—The rolling and manufacture of iron- and steel-work will be inspected at foundries, mills, and shops by

inspectors appointed by, and responsible to, the architect. The general contractor shall include an amount of 80 cents per net ton of iron- and steel-work to meet the cost of such services, and the inspectors shall jointly represent the architect and the general contractor at the places of manufacture, and shall report the progress of the work, and otherwise facilitate the prompt and orderly delivery of satisfactory materials. The inspection, acceptance, or failure to inspect shall in no way relieve the general contractor or the foundries, mills, or shops from their responsibility to furnish satisfactory materials strictly in accordance with the contract, plans, and specifications.

Miscellaneous.—This specification is intended to provide for complete work, including all necessary connections and details requisite for erection, and to develop the full strength of the structure. Such details are to be considered as specified, and are to be provided by the contractor without additional charge.

Apparent discrepancies in plans or specifications must be referred to the architect, whose decision shall be final, and work done without such decision shall be at the contractor's risk.

The architect reserves the right to reject any and all materials or work at any time before the completion of building, if in his judgment either do not comply with the terms of these specifications and good practice, and his decision as to the true intent of plans and specifications shall be final.

The following clauses, applying exclusively to building practice, are extracted from the specifications for structural steelwork used by Messrs. Purdy & Henderson, Consulting Engineers.

Connections.—All connections of beams to beams, beams to columns, columns to columns, and other important connections shall be riveted wherever the character of the connec-

tions will permit. Where rivets cannot be used, tight-fitting bolts may be substituted.

Character of Materials.—All beams and channels and all the column material shall be of steel as hereinafter specified. All connecting angles and plates shall be of steel. All rivets shall be of steel. Tie-rods, bolts, anchors, and lateral ties shall be of wrought-iron. Bearing-plates for beams in masonry, except as specified, bases under the columns, separators, and filler-blocks more than $1\frac{1}{2}$ ins. thick, shall be made of cast-iron.

Beams.—In general, not more than one-eighth ($\frac{1}{8}$) of an inch will be allowed for clearance at each end of beams connecting to beams, and one-fourth ($\frac{1}{4}$) of an inch at the ends of beams connecting to columns. In all cases where possible, the connecting angles used shall be of the same size as those recognized as standard by the Carnegie Handbook, and having the same number of rivets. Beams and girders connecting to columns shall have eight (8) rivets at each end, four (4) in the top flange, and four (4) in the bottom flange, wherever the details of the columns will permit of that number. In all cases the beams must extend as closely as possible to the axis of the columns. The finished floor-line in all cases will be 3 ins. above the tops, and the ceiling-line $1\frac{1}{2}$ ins. below the bottoms, of the 12-in. floor-beams. The height and position of the wall-beams are noted on the sections. Unless otherwise particularly noted, all beams or other long pieces of iron are indicated to their approximate lengths by a single line on the floor-plans.

Columns.—Columns shall be made, in general, in double lengths reaching through two floors as indicated by the section sheet. In general, columns must be connected to columns by splice-plates on the side, riveted to the flanges of the channels with twelve (12) rivets in each column. These plates must be $\frac{5}{8}$ in. thick, except where the metal of the columns connected

is $\frac{3}{4}$ in. thick or more, in which case the splice-plates must be $\frac{1}{2}$ in. thick. Where the outside measurement of one column is less than the other a clearance of more than $\frac{1}{16}$ in. must be taken up with fillers made of bars 3 ins. $\times$ $\frac{1}{16}$ in., punched the same as the splice-plates. All columns will have $\frac{3}{4}$ in. cap-plates. All columns shall be milled at each end to a smooth bearing-surface at right angles to the columns. The point at which the change in section is made is in general 18 ins. above the top of the 12-in. beams. The contractor will be required to furnish the architect with a drawing or schedule showing the heights at which he desires to make these cuts, showing the length of each column and the relation of each cut to the bottom of the regular floor-beams of the floor at the same level, for his approval. The number of rivets required in connections supporting beams must be calculated on a basis of a floor-load of . . . . lbs. per square foot of floor, or on the basis of the full capacity of the beam carrying an evenly distributed load, whichever may require the larger number.

Separators.—Separators must be provided for all double beams; and unless measurements given make it impossible, all separators must be standard.

Tie-rods.—Tie-rods $\frac{7}{8}$ in. in diameter must be provided on all floors, and $\frac{3}{4}$ in. diameter in roof, as shown on plans. These rods must be made with two nuts.

Bolts and Rivets.—Rivets must be calculated for shear at not more than 9,000 lbs. per square inch of section. All rivets must be accurately spaced, and drifting that will be liable to injure the material will not be allowed. Rivet-heads must be located centrally concentric with the neck, and rivets when driven must completely fill the holes. Wherever possible the rivets must be machine-driven. Rivets must be used in all field connections where riveting is possible, and such work must be done to the entire satisfaction of the superintendent in charge. Both bolts and rivets must be $\frac{3}{4}$ in. in diameter

throughout the building, except in special cases where it is necessary to use other sizes.

Bases.—Cast-iron bases must be provided for all columns. These bases must conform to the accompanying drawings, and must be planed smooth on top and to the dimensions given for height. The ribs must be spaced and arranged in each case so that the entire cross-section of the column shall have a direct support from the bottom of the base. The holes for the bolts connecting the columns to the bases must be drilled, after the bases are cast, to exact measurements, which must be obtained when the columns are detailed. These bases must be set by the contractor to exact centre and to exact height, and a variation in height of over $\frac{1}{16}$ in. will not be allowed. They will be bedded in position by the contractor for the masonry.

Temporary Bracing.—If for any reason the masonry in the exterior wall does not follow closely upon the erection of the ironwork, the contractor must put temporary timber braces in to keep the construction of the steelwork plumb until the walls are in place. This must be done to the entire satisfaction of the architects.

Painting.—The covered surfaces, surfaces in contact, and surfaces enclosed, of all parts of riveted members must receive one good coat of . . . . paint, after the pieces are punched and before they are assembled. All finished members must receive one complete coat of paint before they are taken from the shop or exposed to the weather. All surfaces that can be reached must have two coats of . . . . paint after erection. All bolts remaining permanently in the building must be dipped in . . . . paint before being placed in position. All paint must be done on dry surfaces, and preferably warm ones. All dirt or foreign matter of any kind must be removed from the iron before painting. All scale must be removed from finished members before painting the first coat in the shop. All rust

that has accumulated on the material must be removed before painting. The paint used must be the . . . . prepared and mixed by the . . . . Company, of . . . ., ..., and the second coat must have an entirely different color from the first and third coats.

Erection.—Use of iron hammers in driving and bending iron will not be allowed where it can possibly be avoided. Wooden mauls must be used wherever their use is possible, and care must be exercised to prevent the beams and columns from falling in order to protect the metal from heavy shocks.

The structural iron must not be set in advance of the masonry covering, to exceed three stories, unless specifically allowed by the architects. Especial care must be exercised to keep all the columns plumb and in proper line during the erection.

Inspection.*—The use of steel in buildings of ten or more stories, or in manufacturing plants where the floor-loads are heavy and frequently "live" in the sense of causing vibration, has led to more careful specifications as to the quality of material and character of workmanship, to assure which it is the practice of the leading engineers or architects to have the structural frame inspected and tested during manufacture at the foundries, rolling-mills, and shops. This work is generally performed by a firm of engineers who make a specialty of inspection, and who have a number of trained employees permanently located at the principal manufacturing centres, and who, through long experience and working for a number of clients at the same time, are able to perform such inspection efficiently and economically.

It is not feasible for an architect to attempt a similar inspec-

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* For much valuable information on the subjects of Mill, Shop, and Field Inspection, see Chapter XXI, "Inspection of Materials and Workmanship," in Waddell's "De Pontibus."

tion at the building site, because, while he can inspect as to the workmanship, he can form no opinion as to the quality of material. Further, the delays and cost occasioned by errors which have to be corrected at the building are such as to warrant inspection at as early a date as possible in order to avoid them. It is also not feasible for an architect to attempt inspection at the mills and shops himself unless he is prepared to employ several men or else have the inspection incomplete and perfunctory.

The cost of inspection by a responsible and well-equipped firm is not great, and will run from 60 cents to $1.00 per net ton, depending upon the character and weight of the various members. Such cost is properly met by the owner, and may be either provided for directly or, as is frequently done, by a clause in the specifications about as follows:

"The structural iron and steel framework shall be inspected and tested during its manufacture at foundries, rolling-mills, and shops by a competent firm of inspecting engineers, who shall be appointed by the architect and be responsible solely to him, but who shall also represent the owner and the contractor with the view of securing the prompt and orderly delivery of materials in accordance with the contract and specifications. The contractor shall include in his bid an amount equivalent to eighty cents per net ton, which shall be paid monthly for such services."

It is a mistake to sanction cheap terms for inspection. If it is worth doing at all it is worth doing well, and it is better to pay a fair price and have reliable service than to pay less and have the inspection incomplete and slipshod.

Well-performed inspection should include the inspection of all castings at foundries and plates, shapes, etc., at rolling-mills. The inspectors should personally identify the test specimens and conduct the making of tests. Each piece of rolled material should be examined for surface defects, straightness,

and section, and if acceptable should be marked with a special brand—generally a die on a stamping-hammer—and surrounded by a circle of white paint. There should also be resident inspectors located at the manufacturing shops during the entire progress of the work, the theory being that the greatest value of the system is to prevent mistakes and facilitate the work, rather than merely discover errors when it is too late to accomplish satisfactory correction without important delays. Without going into too great detail, shop inspectors should see that all material is straight before and after punching; that holes are reamed where required; that material is assembled correctly before riveting, so that errors in not following plans may be easily corrected; that riveting is tight and of neat appearance, and that all machine-work is accurate and workmanlike. Painting, including the thorough removal of scale and freedom from rust, should receive particular attention. Paint should be known to be the brand specified, and can be analyzed with advantage. Any good paint with pure oil properly applied will prove satisfactory, but there are many methods of adulteration and slighting of work in connection with painting. Shop inspectors should make a final inspection of all members and see that the marking is clear and adequate, and should keep record of the actual weights for comparison with estimated weights. Reports of progress of work should be made weekly, and a final report on completion of manufacture.

With such inspection under intelligent direction much can be done to further the work, not only in preventing and intelligently correcting errors, but in securing the orderly delivery of work as required at the building site. As the steel frame is generally the part of an important building upon which all other work depends, the saving of a few days' delay represents a saving of interest charges which will more than cover the cost of inspection.

The following clauses relating to inspection should be distinctly specified in addition to the matter previously quoted in the forms of specifications given:

Inspection.—Manufacturers should give notice before the commencement of rolling or casting, and reasonable information thereafter; they should give opportunity for inspection by daylight during the regular course of handling of material, or by special handling, and all material should be turned to permit examination on all sides.

Identification.—Each piece of material should be branded with the number of the original furnace heat, except in the case of pieces which will not be under important strain in the structure, when the requirements for such branding may be waived by the inspector. Material from stock should not be used to meet important strains in members in the structure unless identified as above and tested, or the quality assured by undoubted records.

Records.—Manufacturers should furnish the inspectors with records of chemical analyses and press copies of shipping invoices. All records or books giving information as to the quality of material should be open to the inspectors.

Shipments without Inspection.—Shipments made without inspection should be at the risk of the shipper, and if reasonable facilities for the inspection were not provided, the additional cost of subsequent inspection should be borne by the shipper. Any material found to be defective should be immediately replaced, and the engineer or architect may properly reserve the privilege of purchasing such material in the open market at the expense of the shipper. To those who have been kept waiting for material for weeks while an interminable correspondence was carried on, and finally forced to accept unsatisfactory material rather than a greater evil of continued expensive delay, this suggestion should appeal strongly. Shops usually deal with one or two mills with whom they have

credit. They, therefore, are not inclined to buy in the open market.

Relative Value of Detailed Inspection.—The following table * will serve to show the relative values of the details of thorough inspection at rolling-mills and shops, as taught by experience:

	Percentage Values.
Mill Inspection.	
(1) Examination of rolling-mill stock and supervision of methods....	3
(2) Identification of test pieces with furnace melt and material from which it is supposed to be cut	8
(3) Tests made personally by inspectors, including not only tensile tests, but bending and drifting tests; record of latter made by outlining on back of tensile-test blank	8
(4) Chemical analyses investigated and checked	2
(5) Surface inspection of each and every piece of material with identification of accepted material by brand and complete records of accepted and rejected material with description, heat numbers and weight............	20
Shop Inspection.	
(6) Drawings checked for clearance and compared with lists of material............	3
(7) Weights estimated	3
(8) Shop-work supervised during the entire progress, including reinspection of material and detailed inspection of all portions of the work, including patterns and templates, punching, reaming, assembling, riveting, with tests of rivets, machine-work, finishing, painting, marking, weighing, loading, and shipment.......	25
(9) A thorough final inspection covering all important dimensions, matching of field connections, clearances, and all details which will affect the strength or the ease of erection of the structure.	10
Inspection from Main Office.	
(10) Weekly reports showing progress of work............	5
(11) Final report, a concise summary of weekly reports, with rearrangement of test results suitable for file, being a demonstration of exhaustive testing and thorough inspection	10
(12) Personal general supervision by heads of inspecting firm	3
	100

The foregoing branches of inspection work are valuable and necessary. Items 6 and 7 can properly be omitted when the

* From Hildreth & Co., Inspecting Engineers.

checking of drawings and estimating of weights is done by the architect or engineer. Otherwise no modification is to be recommended, although the prices of inspection could possibly be reduced if it were considered advisable to omit any of the other details.

Cheap and poor inspection usually omits items 1, 2, 3 in part, 4, 5 in whole or part, 6, 7, 8, 11, and 12, and includes only 9 and 10 of an unreliable character. The percentage value of such cheap and poor inspection will vary from 10 to 30 as compared to the value of good work.

INDEX.